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February, 1959

Papers

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UNSTEADY FLOW IN CONDUITS WITH SIMPLE SURGE TANKS

E. H. Taylor,[1] A.M. ASCE, Arnold Reisman[2] and Jack W. Ward[3]

ABSTRACT

The basic principles of the operation of simple surge tanks have been known for decades. American and European literature contains many articles devoted to this subject. In spite of the fact that the fundamental flow equations are simple in concept, they are non-linear and hence difficult of solution. The bulk of the existing literature with one relatively recent exception(1) has been concerned with numerical and graphical methods of solution. This paper presents a general solution of the simple tank problem in terms of a single dimensionless parameter for the case of instantaneous and complete load rejection. The mechanical differential analyzer of the University of California was used in obtaining the solution. Seventeen curves covering the range from the case of practically no damping to that of almost critical damping are presented. Whereas nothing new in principle is presented, it is believed that the remarks below constitute a refinement of existing solutions and present numerical data covering a wide range of conditions in concise manner.

INTRODUCTION

The problem under consideration involves the prediction of the water surface elevation in the surge tank, depicted schematically in Fig. 1, as a function of time following rapid and complete cessation of flow in the conduit.

Note: Discussion open until July 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1933 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. HY 2, February, 1959.

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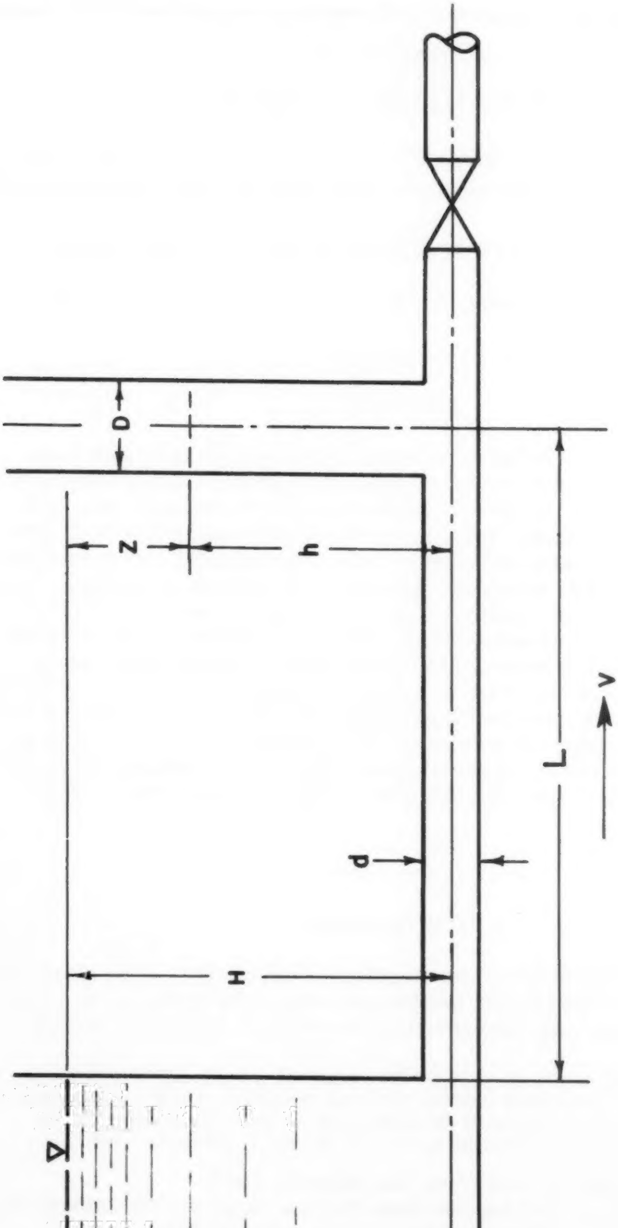

PROFILE DIAGRAM - SIMPLE SURGE TANK

FIGURE I

Theory

The fundamental equation of unsteady flow in the conduit, based on the so-called "rigid water column" analysis, is, in consideration of Fig. 1:

$$H - h - H_{friction} = \frac{L}{g}\frac{dV}{dt} \tag{1}$$

$$\text{or} \quad - z = \frac{L}{g}\frac{dV}{dt} + f \cdot \frac{L}{d} \cdot \frac{V^2}{2g}\,. \tag{2}$$

in which z is measured positively upward. Very simply interpreted, the equation states that the driving head less the friction head is equal to the inertia of the water column in the conduit. It is assumed that the inertia and friction of the water column in the tank are negligible in comparison with those in the pipe.

The continuity condition at the conduit-tank junction requires that

$$aV = A\frac{dz}{dt} \tag{3}$$

for the case of instantaneous and complete closure.
Differentiating (3):

$$a\frac{dV}{dt} = A\frac{d^2z}{dt^2} \tag{4}$$

V and $\frac{dV}{dt}$ are eliminated between (3), (4) and (2) upon arrangement of terms:

$$z + \frac{L}{g} \cdot \frac{A}{a} \cdot \frac{d^2z}{dt^2} + \frac{fL}{d} \cdot \frac{1}{2g} \cdot \left(\frac{A}{a}\right)^2 \left(\frac{dz}{dt}\right)^2 = 0$$

simplifying:

$$\frac{d^2z}{dt^2} + \frac{f}{2d}\frac{A}{a}\left(\frac{dz}{dt}\right)^2 + \frac{ga}{LA}z = 0 \tag{5}$$

To account for damping during flow reversal in the conduit it is necessary to write eq. (5) in the following form.

$$\frac{d^2z}{dt^2} + \frac{f}{2d}\frac{A}{a}\frac{dz}{dt}\left|\frac{dz}{dt}\right| + \frac{ga}{LA}z = 0. \tag{6}$$

This merely precludes the possibility of the damping term having opposite sign from the other terms in which case energy would appear to be added to the system by dissipative forces.

Equation (6) may be rendered dimensionless by the following changes of variables:

Let $\theta = \frac{t}{\tau}$

where $\tau = 2 \cdot \pi \sqrt{\frac{LA}{ga}}$

It is worth noting that τ is the period of undamped oscillations in the conduit-tank system.
Further let

$$\zeta = \frac{z}{z_o}$$

in which z_o is the driving head for steady flow.

$$\text{Then} \quad \frac{dz}{dt} = z_o \frac{d\zeta}{dt}$$

$$\text{and} \quad \frac{d\zeta}{dt} = \frac{d\zeta}{d\theta}\frac{d\theta}{dt} = \frac{d\zeta}{d\theta}\frac{1}{\tau}$$

$$\text{(a)} \quad \frac{dz}{dt} = z_o \frac{d\zeta}{d\theta} \cdot \frac{1}{\tau}$$

$$\text{(b)} \quad \frac{d^2z}{dt^2} = z_o \frac{d^2\zeta}{d\theta^2}\frac{d\theta}{dt} \cdot \frac{1}{\tau}$$

Equation (6) now becomes

$$z_o \frac{d^2\zeta}{d\theta^2} \cdot \frac{1}{\tau^2} + \frac{f}{2d}\frac{A}{a} z_o^2 \frac{1}{\tau_o^2}\frac{d\zeta}{d\theta}\left|\frac{d\zeta}{d\theta}\right| + \frac{ga}{LA}\zeta z_o = 0$$

Multiplying by τ_o^2/z_o

$$\frac{d^2\zeta}{d\theta^2} + \frac{fAz_o}{2da}\frac{d\zeta}{d\theta}\left|\frac{d\zeta}{d\theta}\right| + (2\pi)^2\zeta = 0 \qquad (7)$$

The dimensionless coefficient $\frac{fAz_o}{2da}$, further designated by the symbol ψ, is the damping factor for the normalized equation. Finally

$$\frac{d^2\zeta}{d\theta^2} + \psi\frac{d\zeta}{d\theta}\left|\frac{d\zeta}{d\theta}\right| + (2\pi)^2\zeta = 0. \qquad (8)$$

The initial conditions, corresponding to the instant the valve is closed are:

$$\zeta = -1, \quad \frac{d\zeta}{d\theta} = \frac{2\pi}{\sqrt{\psi}}, \quad \frac{d^2\zeta}{d\theta^2} = 0$$

Computer Solutions

A diagram of the differential analyzer mechanization is shown in Fig. 2. Our remarks will be restricted to this particular problem since the theory of the differential analyzer has been presented in the literature.[2,3] For the most part, standard techniques were used in obtaining solutions to Eq. (8). The equation was written in the form

$$\frac{d^2 \zeta}{d\theta^2} = -\psi \frac{d\zeta}{d\theta}\left|\frac{d\zeta}{d\theta}\right| - (2\pi)^2 \zeta$$

and the right-hand side of this equation constructed by repeated integration, constant multiplication, and summation. The use of an integrator (No. 2) to multiply by a constant is standard where a large number of parameter values are used. A clutch or "disconnect" between integrators 3 and 1 in order to introduce a factor of -1 whenever $\frac{d\rho}{d\theta}$ became negative (so as to obtain $\left|\frac{d\rho}{d\theta}\right|$) was the only special feature involved. Seventeen solutions to eq. (8) as determined by the above technique are presented in Figs. 3, 4 and 5. A dimensionless plot indicating the height and time of the first maximum surge is given in Fig. 6.

CONCLUDING REMARKS

It was found possible to express the differential equation describing the unsteady flow in a surge tank system during a sudden outflow cessation in terms of dimensionless variables and a single dimensionless "damping" parameter. This equation was solved mechanically and the results are presented in a form which makes possible the determination of the maximum surge, the time of its occurence and an approximation to the time of complete cessation of water pendulation, for most simple surge tank system configurations.

The computer solutions were substantiated within 1% accuracy by numerical analysis of a typical system.[4] An experimental model study of the University of California, Los Angeles (unpublished) likewise indicated substantiation of the computer solutions to the extent that the height and time of the first maximum surge were found to be in excellent agreement with the above theory. However, the rate of damping as observed in actual flow has been tentatively found to be at variance with that predicted by the theory. In addition, the conditions of stability of a simple tank system were found experimentally also to be in disagreement with the Thoma criterion which is widely quoted in the literature.[5,6] These latter matters are currently subject to investigation.

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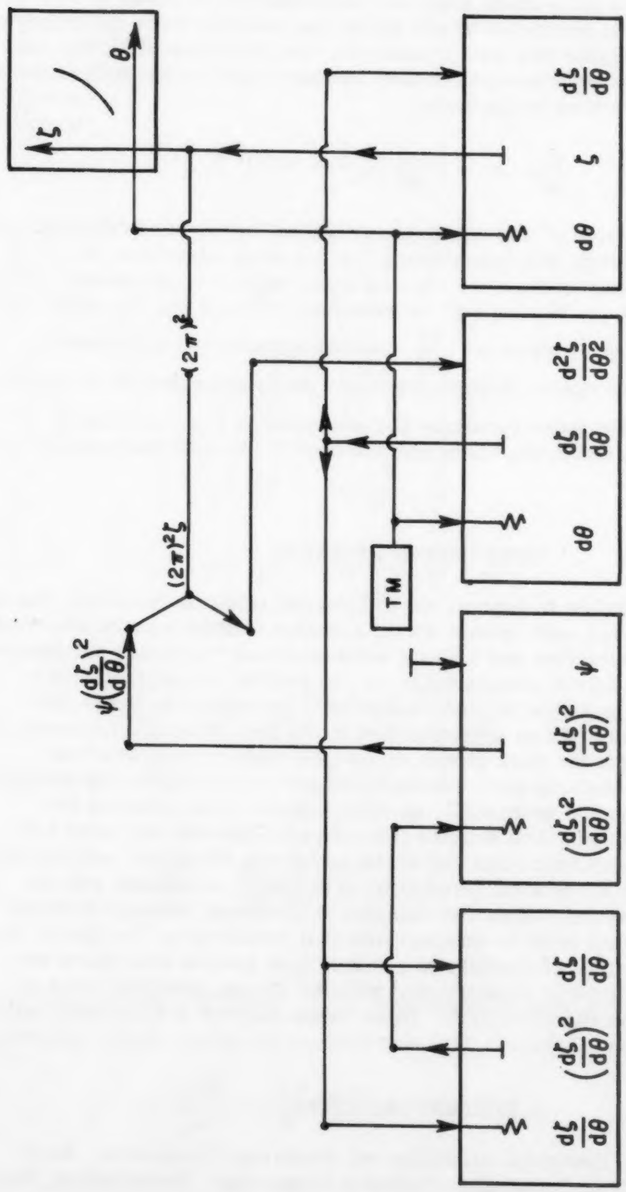
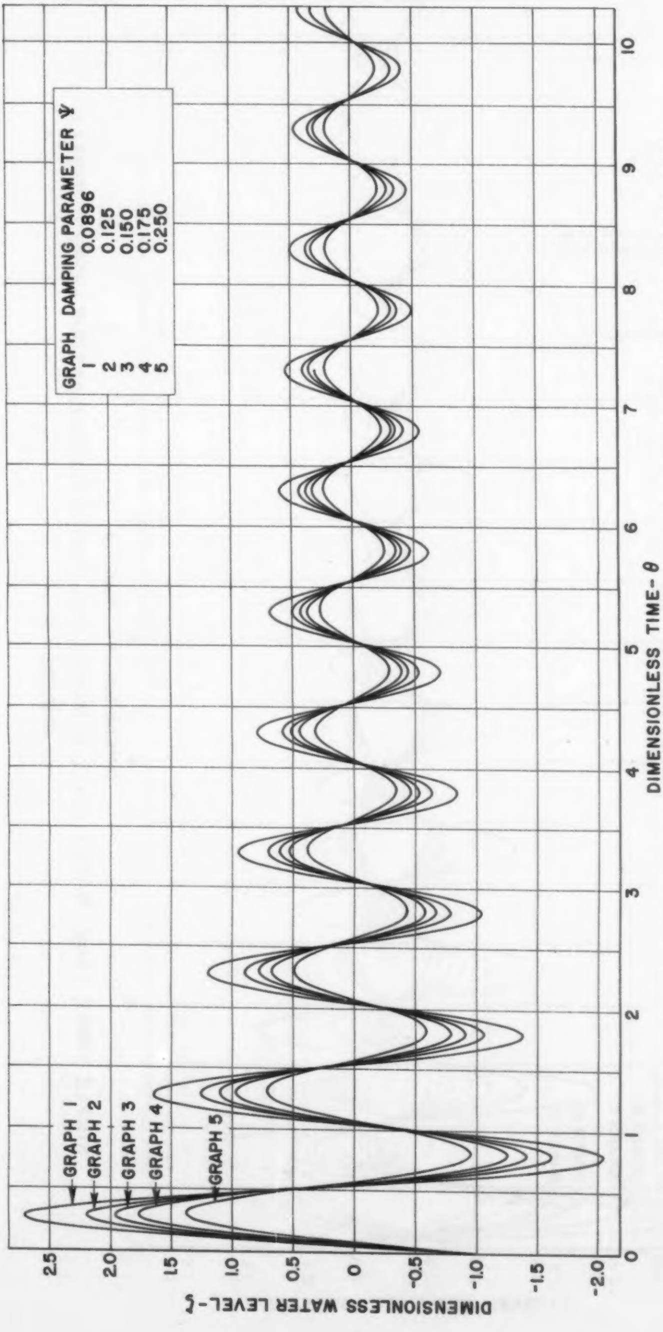


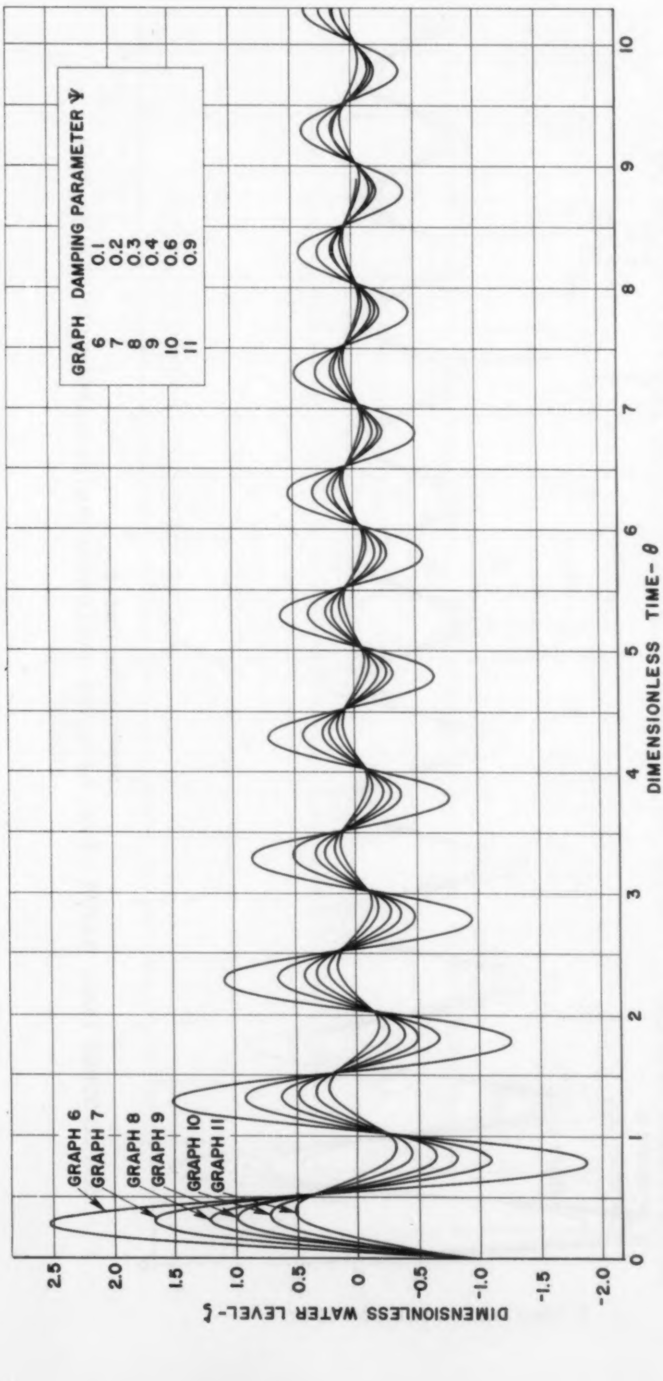
MACHINE BLOCK DIAGRAM. SOLUTION OF SIMPLE SURGE TANK EQUATION ON THE MECHANICAL DIFFERENTIAL ANALYZER

FIGURE 2



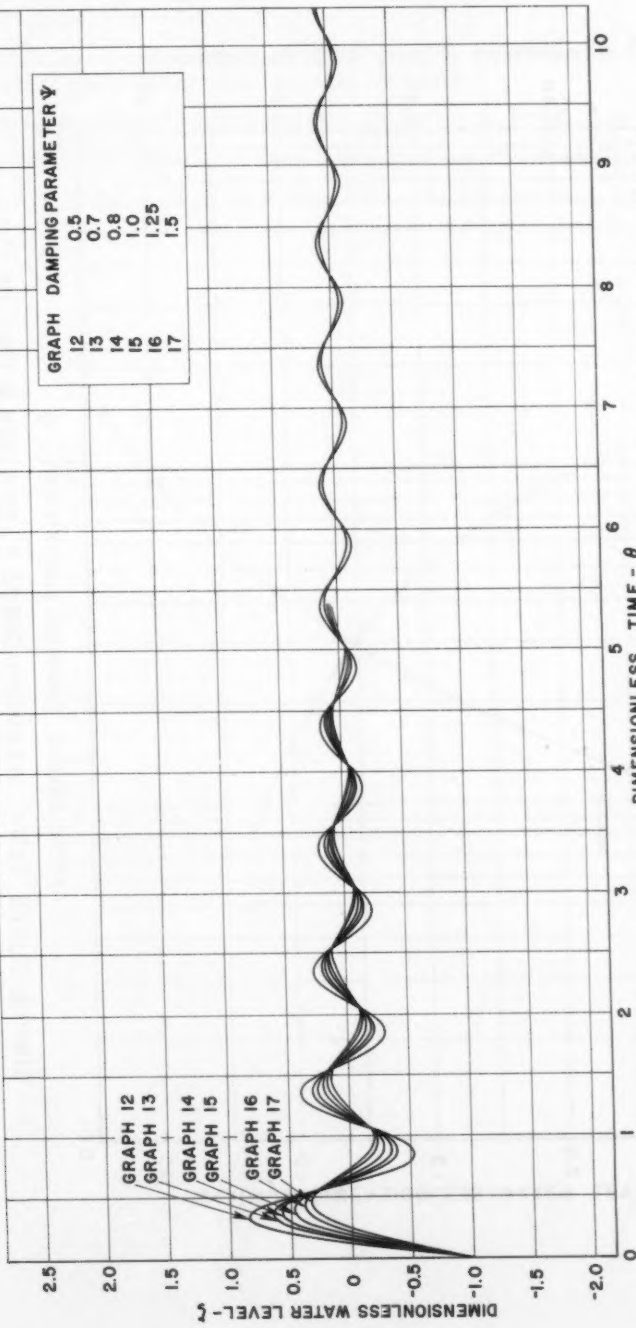
SIMPLE SURGE TANK. WATER LEVEL BEHAVIOR (FOLLOWING INSTANTANEOUS VALVE CLOSURE)

FIGURE 3



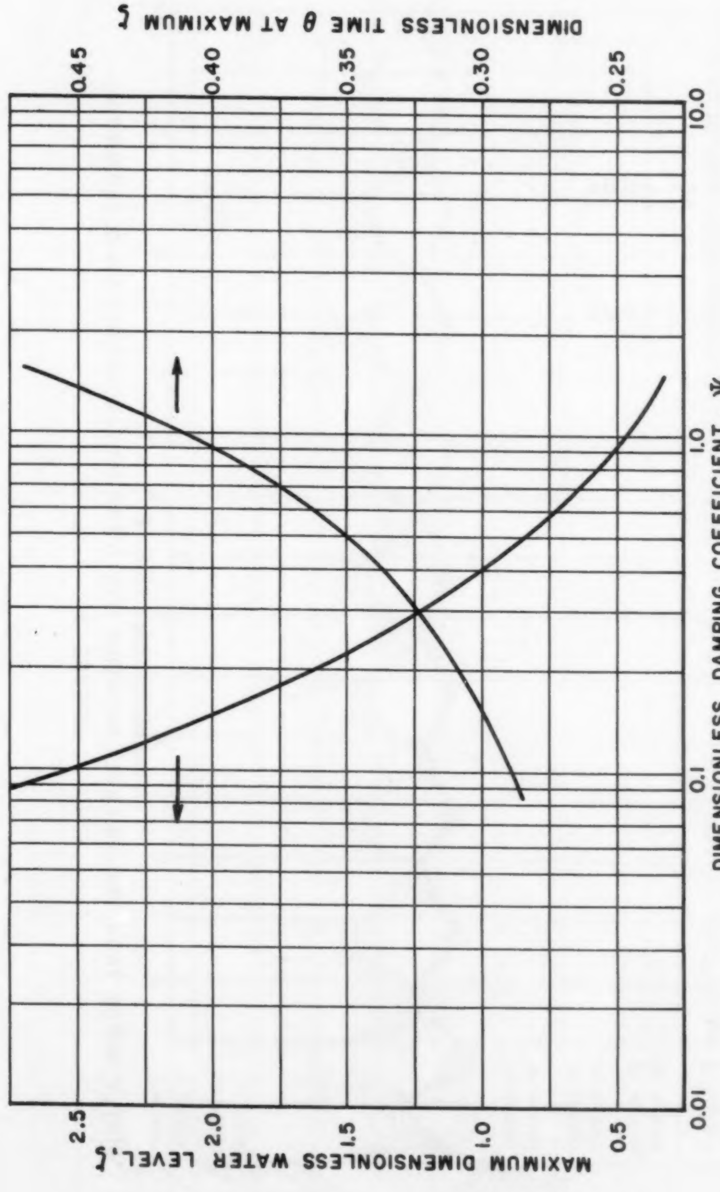
SIMPLE SURGE TANK. WATER LEVEL BEHAVIOR (FOLLOWING INSTANTANEOUS VALVE CLOSURE)

FIGURE 4



SIMPLE SURGE TANK. WATER LEVEL BEHAVIOR (FOLLOWING INSTANTANEOUS VALVE CLOSURE)

FIGURE 5



SIMPLE SURGE TANK. MAXIMUM SURGE ELEVATION & TIME OF ITS OCCURRENCE VERSUS DAMPING COEFFICIENT Ψ, DIMENSIONLESS

FIGURE 6

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Journal of the
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EVAPORATION OF LAKE ONTARIO[a]

Ira A. Hunt, Jr.[1]

SYNOPSIS

There is definite evidence that the precipitation falling over the surface of the Great Lakes is considerably less than the precipitation falling on adjacent land areas. Evaporation computed by the water budget method is reduced in the same amount as the precipitation. The annual amount of net evaporation from Lake Ontario computed by the water budget and mass transfer methods is of the order of 2.0 feet.

INTRODUCTION

The advent of the St. Lawrence Seaway projects has stimulated the diverse interests of landowners, shippers, economists, and engineers, all of whom have been concerned about the levels of the Great Lakes for some time. Yet, despite their attentive concern, many are not aware that the outflow of the St. Lawrence River and, thus, the water levels of Lake Ontario, have been completely regulated by the Iroquois Dam control structure since 1958.

It is the responsibility of the Corps of Engineers, U. S. Army, to ascertain for the United States that the regulation of Lake Ontario provides optimum benefits for navigation, power, and riparian interests. To this end, the U. S. Lake Survey has been studying techniques of forecasting supplies to the Great Lakes for future use in effecting the most beneficial regulation plans. These studies determined that, of all the hydrologic factors affecting the levels of the Great Lakes, evaporation was the factor about which the least was known.

Evaporation is the process by which water is converted to vapor. The molecules that make up water are in continuous motion. Some molecules of water have sufficient momentum to break through the surface and enter the

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a. Presented to the February 1958, ASCE Convention in Chicago, Ill.

1. Maj., U. S. Lake Survey, Corps of Engrs., Detroit, Mich.

air. The rate at which these water particles leave the water and enter the adjacent air depends upon the heat supply of the water and the condition of the air. Concurrently, molecules of water vapor in the air strike and enter the water; that is, they condense. In hydrology, evaporation is considered to be the net loss of water mass.

There are several methods of computing evaporation. For the Great Lakes, the water budget method has been utilized most frequently. More recently, evaporation has been correlated with the vertical distribution of moisture in the air and the intensity of turbulent mixing. This is the so-called mass transfer theory. Another, called the energy budget method, takes into consideration solar radiation, atmospheric radiation, advected energy, sensible heat transfer, and other factors which affect the total amount of heat available to convert water into vapor. Computing evaporation by the water budget and mass transfer methods will be discussed herein.

Water Budget Method

The water budget method of determining evaporation on Lake Ontario is simply a matter of solving the general hydrographic equation for evaporation:

$$E_{1-2} = S_{1-2} + R_{1-2} + P_{1-2} + I_{1-2} - O_{1-2} + U_{1-2} \tag{1}$$

where

E_{1-2} is the evaporation from the surface of Lake Ontario during the interval 1 to 2.

S_{1-2} is the change in the mean level of Lake Ontario during the interval 1 to 2.

R_{1-2} is the runoff of water from the land surface draining into Lake Ontario during the interval 1 to 2.

P_{1-2} is the precipitation falling directly on the surface of Lake Ontario during the interval 1 to 2.

I_{1-2} is the inflow to Lake Ontario from Lake Erie, or the flow of the Niagara River plus the Welland Canal, in the interval 1 to 2.

O_{1-2} is the outflow from Lake Ontario, or the flow of the St. Lawrence River, in the interval 1 to 2.

U_{1-2} is the sum of all factors affecting the rise or fall of the level of Lake Ontario not contained in the other factors of the equation.

Although this method of determining evaporation is simple, the determination of the component factors which make up the equation is quite difficult. In studying Lake Ontario, the amount of hydrological and meteorological data available in the period prior to 1934 was limited, so a twenty-year period of record from 1934 through 1953 was used. Evaporation was computed in monthly units because the majority of data was available on a monthly basis and because it was intended that the results of the study would be used in regulation plans for Lake Ontario where a thirty-day forecast of the water levels is desired.

Normal changes in levels of Lake Ontario

The U. S. Lake Survey, in coordination with the Water Resources Branch, Department of Northern Affairs and National Resources, Canada, has determined Lake Ontario end-of-month levels. From these data, the average monthly changes in Lake Ontario levels for the period 1934 to 1953 were computed and are tabulated in Column 2, Table 1.

In the period of record, 1934-1953, the all-time mean annual low (1860-1957) of 243.54 feet was established in 1935, and the all-time annual high (1860-1957) of 248.01 feet was established in 1952. During this maximum secular climb of Lake Ontario, the average lake levels rose during the period January through May and fell during the period July through December; the

TABLE 1

Evaporation in Feet Computed by Water Budget Method
Period of Record, 1934-1953

$$E_{1-2} = S_{1-2} + R_{1-2} + I_{1-2} - O_{1-2} + P_{\text{over water}_{1-2}}$$

1	2	3	4	5	6	7
MONTH	S_{1-2}	R_{1-2}	$I_{1-2} - O_{1-2}$	$P_{\text{over land}_{1-2}}$	$P_{\text{over water}_{1-2}}$	E_{1-2}
JANUARY	- .15	.46	- .32	.23	.20	.19
FEBRUARY	- .16	.41	- .32	.20	.17	.10
MARCH	- .57	.79	- .36	.25	.20	.06
APRIL	- .60	.97	- .50	.23	.17	.04
MAY	- .27	.59	- .48	.27	.20	.04
JUNE	.00	.30	- .48	.24	.18	.00
JULY	.23	.19	- .50	.28	.19	.11
AUGUST	.46	.14	- .46	.22	.14	.28
SEPTEMBER	.38	.15	- .41	.28	.20	.32
OCTOBER	.36	.20	- .38	.23	.20	.38
NOVEMBER	.15	.29	- .32	.25	.23	.35
DECEMBER	.06	.36	- .30	.24	.22	.34
TOTAL	-0.11	4.85	-4.83		2.30	2.21

maximum monthly rises occurred in March and April, and the maximum monthly declines occurred in August and September.

Runoff

The runoff of precipitation from the drainage basin of Lake Ontario is an extremely important factor in analyzing lake level fluctuation. Preliminary studies have shown that there is a small lag from the time of precipitation until the runoff reaches the lake. Although several of the tributary basins have a great number of small lakes and the basin of the Trent River is partially regulated, there does not appear to be any marked degree of storage in the Lake Ontario basin. Of the 34,795 square miles in the basin, the lake surface consists of 7,520 square miles and the land drainage area consists of 27,275 square miles; therefore, it is not difficult to visualize why runoff is so important.

The majority of runoff from the land area into Lake Ontario is in the form of stream flow from the major rivers tributary to the lake. The files of United States and Canadian agencies were examined for available stream flow records. Usable records for nine major rivers which include 16,500 square miles or 64 per cent of the Lake Ontario land area (excluding that area tributary to the Niagara River above Niagara Falls) were found. Drainage areas with no available records were incorporated into the near drainage areas with surface records, see Figure 1.

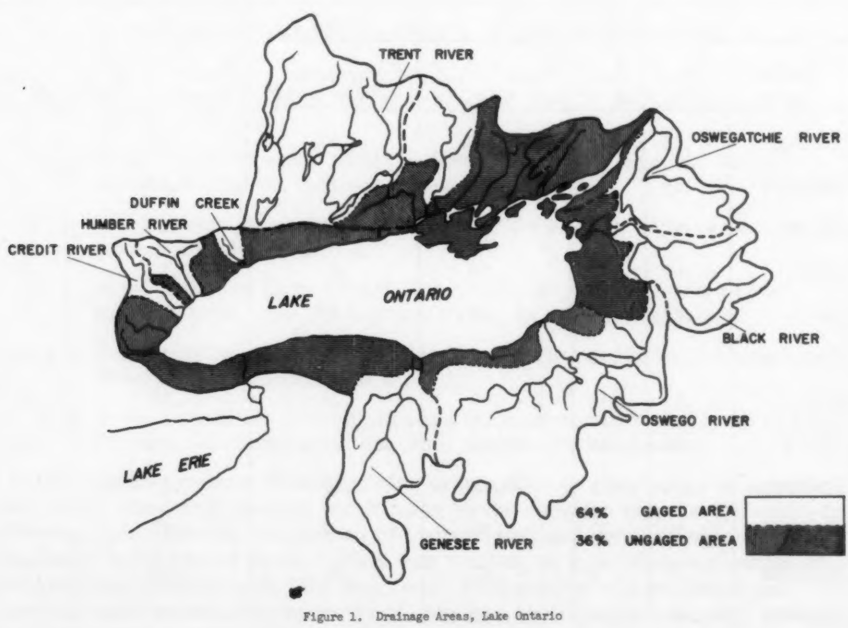

Figure 1. Drainage Areas, Lake Ontario

The areas for which gaged records were not available were divided almost evenly between upland and coastal regions. Although there is not much

variation in the topography of the Lake Ontario Basin, except in the eastern portion where mountain foothills are located, care was taken to incorporate ungaged areas with the gaged areas that had the same general topographical characteristics. For instance, the ungaged area in the coastal region of northern Lake Ontario was incorporated with Duffin Creek which also lies within the coastal region. The ungaged upland regions east of the Moira River were incorporated with the Moira River basin. The size of the gaged drainage areas of the nine rivers and the areas incorporated with the records of these rivers are given in Table 2. The Credit and Humber Rivers and Duffin Creek had limited records and the statistics of flow were mathematically extended to the period 1934-1953.

The average monthly runoff for the lake basin measured in feet on Lake Ontario for the period 1934-1952 is given in Column 3, Table 3.

Although there are certain to be some errors in measuring the total runoff, the gaged runoff is the most accurately measured variable in the hydrologic cycle for, instead of being a point measurement within a large area (such as the measurement of precipitation), the gaged runoff effectively integrates the entire area from which the measured flow originates.

Runoff is a complex phenomena which depends upon many variables. The observed monthly runoff for the Lake Ontario Basin is a function of the supply of water available for runoff and the losses which prevent the supply from reaching the lake. The supply of water available during a month is directly related to the precipitation during that month, the interflow, ground water, and channel storage, and the water equivalent of the snowpack; if any. The losses are due to interception, depression storage, evapotranspiration, and replenishment of soil moisture.

Indexes of these most important variables affecting runoff were determined and a mathematical correlation of historical record with these indexes were obtained which gave results of rather high correlation.(1)

The relation of runoff to precipitation proved to be a useful tool in gaining an insight into the runoff from the basin. This relationship for each of the drainage areas and for the entire basin is given in Table 3. Some very interesting results are shown. One, is the extremely large runoff factors for the months of March and April. This demonstrates conclusively that a knowledge of the storage of precipitation in the form of snow and the subsequent snow melt is of vital importance if a complete understanding of spring runoff is to be obtained. Another, is the magnitude of the yearly runoff factor for the Lake Ontario basin. The value of 48.0 per cent is appreciably higher than the value of 30.7 per cent obtained by John R. Freeman in his study, "Regulation of Great Lakes."

Precipitation over Lake Ontario

The amount of precipitation over the lake affects the lake level immediately. Heretofore, the weighted precipitation catches recorded at selected weather stations on the perimeter of the lake have been regarded as the precipitation falling over the lake. The perimeter precipitation shows a marked reduction from the precipitation falling over the land drainage area, yet it seems that, due to the cooling effect of the lake in the summer and the fact that there are no diurnal heating or orographic effects, the precipitation falling over the lake is much less than indicated by the perimeter stations.

TABLE 2

Basin Areas of Tributary Runoff into Lake Ontario

Basin	Station	Gaged Area	Incorporated Area	Total Area
Oswegatchie River	Near Heuvelton, New York	973	1,735	2,708
Black River	At Watertown, New York	1,876	1,190	3,066
Oswego River	At Lock 7, Oswego, New York	5,121	503	5,624
Genesee River	At Driving Park Avenue, Rochester, New York	2,467	1,908	4,375
Credit River	At Erindale, Ontario	323	495	818
Humber River	At Weston, Ontario	323	166	489
Duffin Creek	Near Pickering, Ontario	112	1,030	1,142
Trent River	Near Campbellford, Ontario	4,306	821	5,127
Moira River	Near Foxboro, Ontario	1,038	1,537	2,575
	TOTAL	16,539	9,385	25,924

TABLE 3

Ratio, Runoff to Precipitation Lake Ontario Basin
Period of Record, 1934-1953

Month	RIVER BASIN									Lake Ontario Basin
	Genesee	Oswegatchie	Moira	Black	Oswego	Trent	Duffin	Humber	Credit	
January	.68	.67	.35	.72	.71	.48	.44	.27	.45	.59
February	.72	.69	.31	.61	.68	.53	.86	.58	.66	.62
March	1.13	1.25	.82	1.10	.86	.50	1.19	.85	1.04	.92
April	.97	1.60	1.86	1.60	.93	.97	.73	.74	.84	1.17
May	.51	.77	.76	.87	.56	.58	.29	.27	.28	.61
June	.29	.40	.41	.43	.36	.29	.28	.19	.31	.34
July	.18	.23	.13	.27	.20	.14	.18	.09	.18	.19
August	.17	.20	.09	.27	.17	.15	.20	.09	.12	.17
September	.15	.17	.04	.26	.15	.13	.16	.06	.11	.15
October	.22	.35	.09	.42	.20	.22	.30	.10	.16	.24
November	.31	.50	.18	.55	.31	.12	.29	.13	.20	.30
December	.52	.54	.25	.56	.52	.30	.31	.21	.26	.43
Yearly Average	.49	.61	.44	.64	.47	.37	.44	.30	.38	$\boxed{.48}$

In late years there have been several concerted attempts to determine if the depth of precipitation falling over large bodies of water is of the same order of magnitude as the depth falling over the adjacent or surrounding land areas. In the Great Lakes area, precipitation has been recorded at St. James, Beaver Island, in northeastern Lake Michigan, for twenty-five years during the period 1911-1956. A comparison of the precipitation falling at Beaver Island with the precipitation falling on the adjacent land areas indicates that not only is the depth of precipitation less over Beaver Island, but that there is a seasonal variation.

Northeastern Lake Michigan studies

To investigate the theory that the depth of precipitation falling over large bodies of water is less than the depth falling over adjacent or surrounding land areas, a study of precipitation over northeastern Lake Michigan was instituted by the United States Lake Survey. The data for the study were obtained jointly with the U. S. Weather Bureau and with the cooperation of the U. S. Coast Guard. Storage precipitation gages were installed on six islands during November 1952, by the U. S. Weather Bureau, see Figure 2. These special island storage gages were inspected and the catch was measured semiannually. The results of the catch on the special island gages have been compared with a network of ten shore control stations.

The topography of the six islands varies tremendously and the ground cover and exposure at the gage sites also differ greatly; yet, the catch at

Figure 2. Precipitation Gage Locations, Northeastern Lake Michigan

five of the six island stations for the period November 1952 to November 1957, compares very favorably with that of St. James, Beaver Island. The average precipitation of the five large island stations for the five year period is 103 per cent of the precipitation recorded at St. James. However, the catch at Ile aux Galets, the smallest of the six special island stations with an area of 2.9 acres, was appreciably less than the catch at the other island stations. Ile aux Galets is a very small flat sand spit and is completely devoid of vegetation, whereas the other island stations are fairly heavily vegetated and relatively large in area.

The greatest difference in precipitation between Ile aux Galets and the other island stations is in the winter period. This is believed to be caused by the extreme exposure of the gage, which reduces the amount of catch of snow.

The records during the summer period show that the depth of precipitation at Ile aux Galets is approximately the same as for South Fox Island. The average precipitation of Ile aux Galets and South Fox Island during the period May through October, 1952-1957, when comparable data are available, is 93 per cent of that recorded at St. James. This is reasonable, because Beaver Island is large enough for definite diurnal heating to occur and the convective currents generated could cause additional precipitation as compared with the smaller islands.

Nevertheless, to determine if the size and vegetative cover of the island had a definite effect on the amount of precipitation, an additional gage was installed in October, 1956, on Shoe Island, another sand spit 2.3 acres in size. Figure 3 shows the gage location on Shoe Island. The results of the first year of operation corroborate the reduced catch on Ile aux Galets, indicating that the rainfall over the small islands, and consequently over the lake, is reduced from that recorded on Beaver Island.

It is fully realized that five years and less are very brief periods on which to base specific conclusions regarding precipitation. But, general trends can be ascertained and approximate relationships can be determined. On the other hand, the twenty-five years of record at St. James are much more reliable. The ratio of precipitation recorded at St. James to that recorded over the surrounding land area is given in Column 2, Table 4. These ratios show a definite reduction in rainfall, with the greatest reduction occurring in the summer months. This is what may be expected, because there are fewer frontal storms in the summer months and those that pass over the lake are dampened due to the cooling effect of the lake surfaces. The convective type precipitation is pronounced over land during the summer period. In the winter, there are more frequent passages of frontal storms, no dampening effect by the lake, and no squall-type storms. Therefore, one would expect the precipitation over the lake to approximate that occurring over the land. The difference in the air and water temperature in the vicinity of Beaver Island is illustrated in Figure 4. It can be seen that the average monthly air temperature is higher than the average monthly water temperature during the period April through August, accounting in part for the reduced rainfall over the lake during this period.

In a previous paragraph it was stated that the precipitation over the lake was indicated to be less than that recorded at St. James. From the records of South Fox Island, Ile aux Galets, and Shoe Island, it seemed reasonable and conservative to assume the reductions shown in Column 3, Table 4. Thus, it

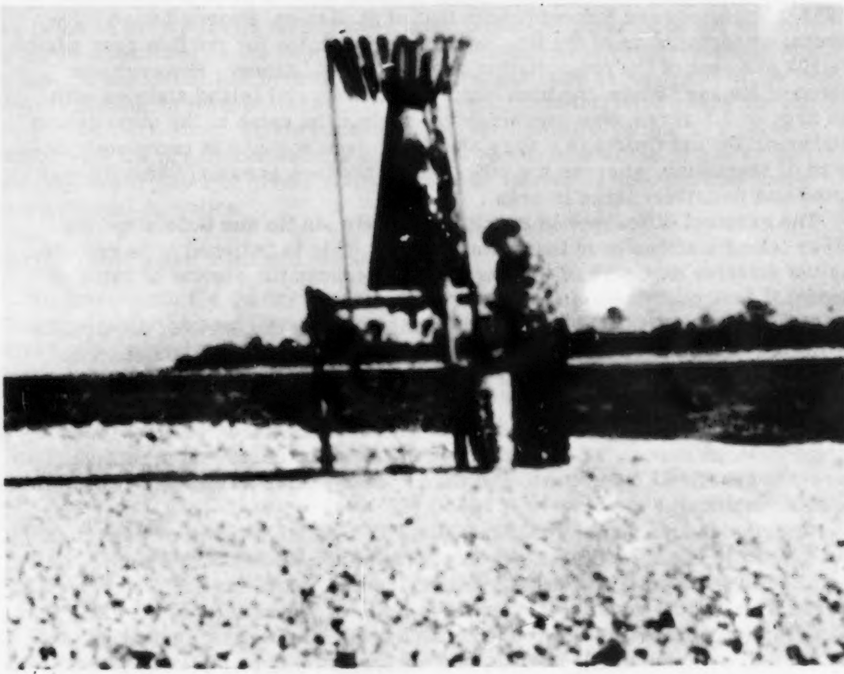

Figure 3. Precipitation Gage, Shoe Island

was assumed that the product of the two ratios, the ratio of St. James to land area and the ratio of small island stations to St. James, would give an approximate solution to the reduction in precipitation falling over northeastern Lake Michigan. A plot of a smoothed curve drawn through the values of the product of the two ratios, or the ratio of precipitation over water to precipitation over land, appears on Figure 5, and the "smoothed" monthly values used to reduce land precipitation over northeastern Lake Michigan are given in Column 5, Table 4.

In his paper entitled, "The Influence of Lakes and Urban Areas on Radar Observed Precipitation Echoes," Bulletin of American Meteorological Society, J. E. Pearson gives the conclusion that, "Observation of radar film of echoes in the vicinity of or over Lake Michigan revealed evidence that the lake, in general, discourages the formation of air mass showers." This fact is definitely evidenced by the outline of radar echoes as depicted in Figure 6. In addition, data collected by the Jacksonville District, Corps of Engineers, at precipitation stations on Lake Okeechobee, the second largest fresh water lake in the United States with an area of 730 square miles, reveal a reduction in rainfall over the lake compared with the catches on the perimeter of the lake.

TABLE 4

Precipitation Ratios, Northeastern Lake Michigan

1	2	3	4	5
Month	St. James to Surrounding Land	Small Islands to St. James	Small Islands to Surrounding Land	"Smoothed" Small Islands to Surrounding Land
January	.90	1.00*	.90	.89
February	.86	1.00*	.86	.85
March	.76	1.00*	.76	.79
April	.80	.95*	.76	.76
May	.81	.93	.75	.76
June	.85	.93	.79	.74
July	.74	.93	.69	.69
August	.64	.93	.60	.62
September	.80	.93	.74	.72
October	.96	.93	.89	.85
November	.91	1.00*	.91	.90
December	.88	1.00*	.88	.90
Yearly Average	.83			.79

*Hypothetical

The U. S. Weather Bureau office in Key West has approximated their rainfall as 60 per cent of that falling on the Florida mainland. Thus, there is conclusive evidence that the precipitation falling over large bodies of water is reduced.

One may reasonably ask if the ratios obtained for northeastern Lake Michigan are applicable to Lake Ontario. Evidence indicated that approximately the same ratios are applicable. Generally, the same prevailing winds hold for the whole Great Lakes region. A comparison of precipitation at the perimeter stations with the land drainage area stations of Lake Michigan and

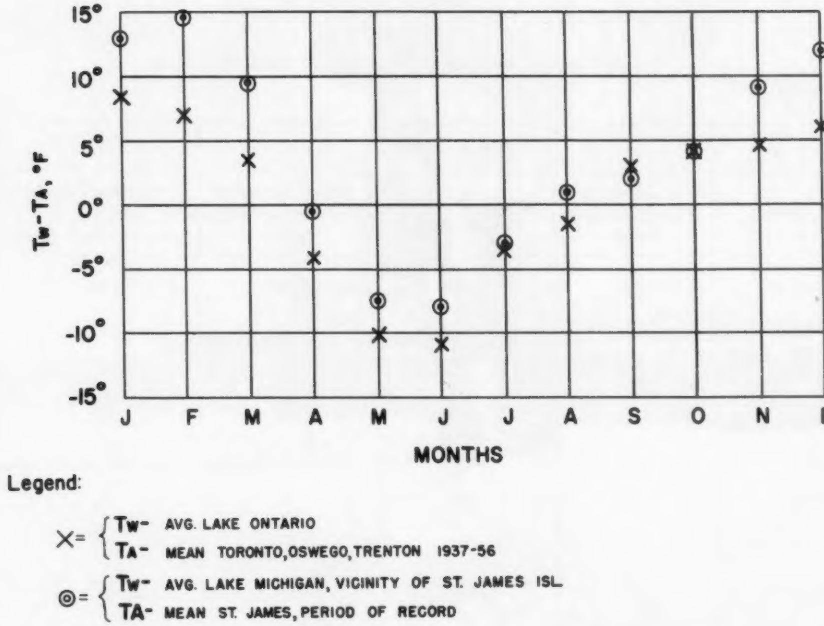

Figure 4. Average Monthly Air-Water Temperature Differentials in Degrees Fahrenheit, Lakes Michigan and Ontario

Lake Ontario showed that the ratios are remarkably the same. As shown in Figure 7, the reduction in precipitation over both lakes follows the same pattern. Furthermore, a comparison of the average monthly water temperature minus the average monthly air temperature revealed that almost exactly the same pattern is followed by both Lakes Michigan and Ontario, see Figure 4, above. This indicated that the lakes discourage the formation of air mass showers during the same period of the year. Accordingly, factors developed for reducing the amount of precipitation falling over northeastern Lake Michigan will apply to Lake Ontario.

The average monthly precipitation falling over the Lake Ontario land drainage area is constant, varying but slightly from the average monthly mean of 0.24 foot. The average monthly over-land precipitation is shown in Column 5, Table 1, above. This average monthly precipitation over land, multiplied by the ratios previously computed for reducing the precipitation over land to the precipitation over water Column 5, Table 4, gave the average monthly depth of precipitation falling on Lake Ontario, see Column 6, Table 1.

Inflow to Lake Ontario from Lake Erie

The inflow to Lake Ontario from Lake Erie is the flow of the Niagara River plus the diversion of water through the Welland Canal in Canada. The

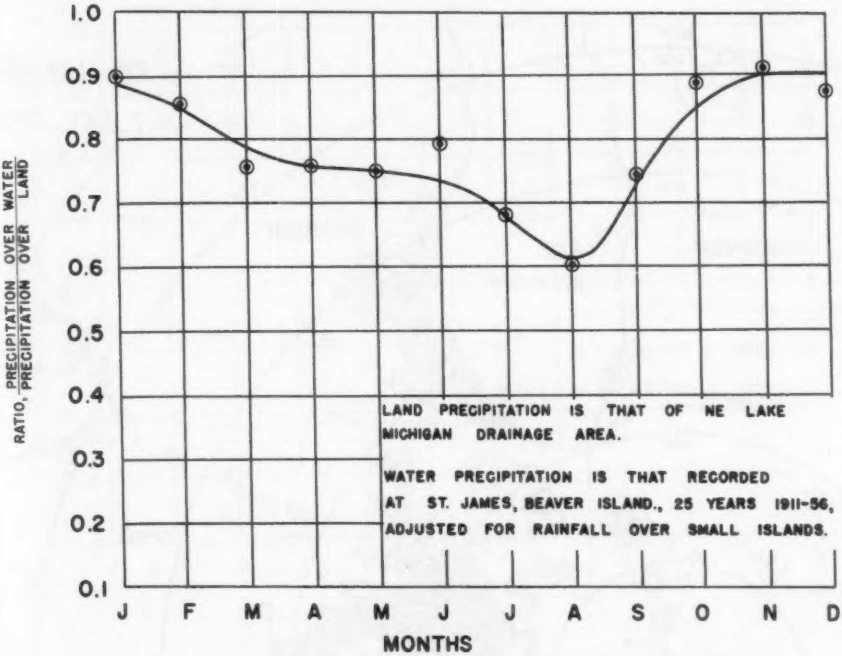

Figure 5. Ratios, Precipitation Over Water to Precipitation Over Land

Welland Canal diverts water for navigation and power from Lake Erie at Port Colborne and returns it to Lake Ontario at Port Weller and Port Dalhousie. The Niagara River flows used in this study were computed by the new Morrison Street rating curve. The Morrison Street gage is located in the Maid of the Mist Pool below Niagara Falls and was the only gage on the Niagara River which was comparatively free from ice and weed effects during the period of record. It is interesting that the flows used in this study differ from the published flows of the Niagara River which occurred at high Lake Erie levels by as much as 10,000 cfs per month. Revision of the published Niagara River flows is being studied by the United States Lake Survey.

Outflow of the St. Lawrence River

The Coordinating Committee on Great Lakes Basic Hydraulic and Hydrologic Data(2) has accepted the premise that the Rapide Plat weir is the best available meter of the flow of the St. Lawrence River and that the flow over the weir can be correctly measured by utilizing the stage at the Lock 25 gage. The committee, therefore, adopted the Lock 25 stage-discharge relationship as the basic relationship for the determination of the river flow. The Lake Ontario outflows utilized in this study are those adopted by the Coordinating Committee on Great Lakes Basic Hydraulic and Hydrologic Data.

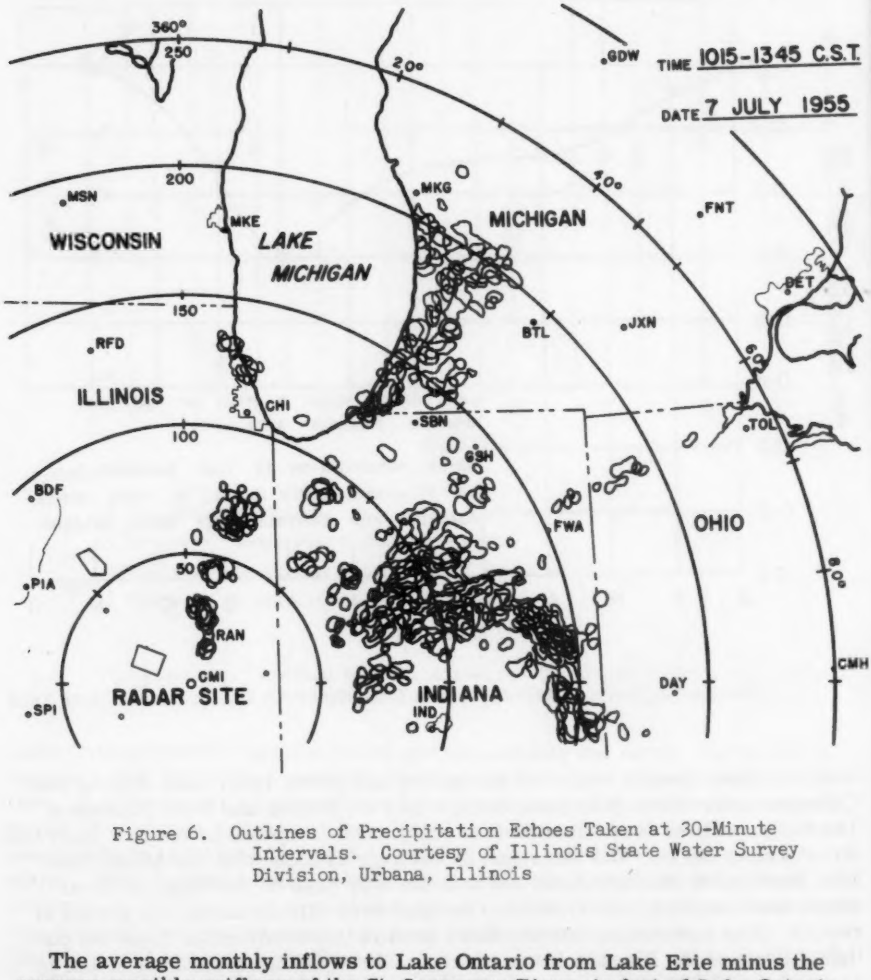

Figure 6. Outlines of Precipitation Echoes Taken at 30-Minute Intervals. Courtesy of Illinois State Water Survey Division, Urbana, Illinois

The average monthly inflows to Lake Ontario from Lake Erie minus the average monthly outflows of the St. Lawrence River, in feet of Lake Ontario levels, for the period 1934-1953, are shown in Column 4, Table 1, above. These values do not vary greatly and there is excellent correlation between the inflows and outflows when they are analyzed statistically.

Other factors

All other factors which affect the amount of evaporation computed by a water budget equation not previously discussed were included in the item U_{1-2}. The most important of these factors were the ground water flow and the thermal expansion of water.

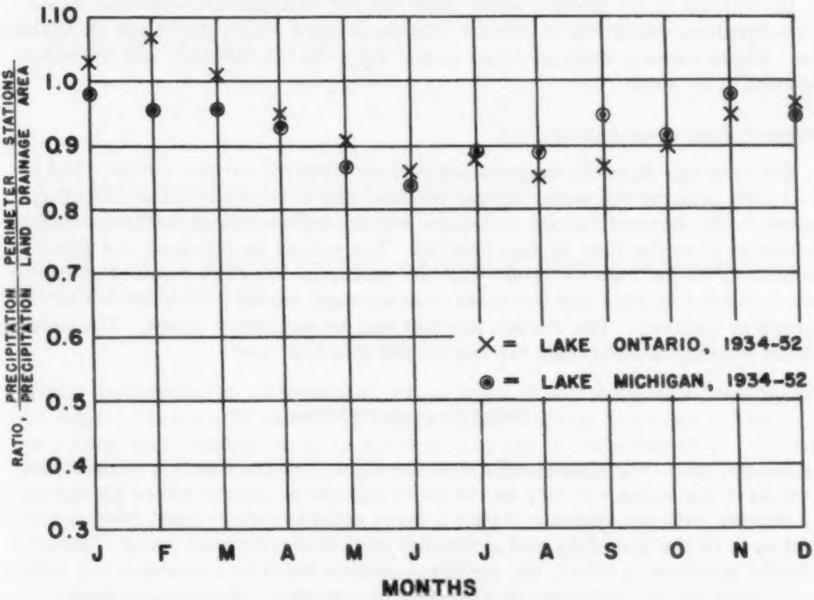

Figure 7. Ratio, Precipitation at Lake Perimeter Stations to Precipitation Over Land, Lakes Michigan and Ontario

Ground water flow

The average level of Lake Erie is 572.3 feet, whereas, the average level of Lake Ontario is 246.0 feet. It was possible that there might be a considerable underground flow between the two lakes due to the very large potential head. This matter was discussed with members of the U. S. Geological Survey and the consensus was that there was not an appreciable ground water flow into Lake Ontario.

Thermal expansion of water

As a mass of water is heated and cooled, it will expand and contract. The greater the depth, the larger the thermal expansion and contraction. The average depth of Lake Ontario is 264 feet. However, as already shown, a definite thermocline exists in Lake Ontario. This thermocline varies in depth both with time and position, but the maximum depth does not exceed 50 feet, below which the water is generally at a temperature of 39° F. The maximum monthly average water temperature difference between two consecutive months was 11° F, see Table 4 above. The lake expands in a vertical direction only, and the expansion is a function of the change in average density of the water above the thermocline. The maximum amount of expansion and contraction is about .01 foot per month. Consequently, the thermal expansion of

water was not considered in the water budget equation.

In addition to the ground water flow and the thermal expansion of water, consideration was given to other probable factors which might not be negligible. These factors were grouped under U_{1-2} in Equation (1) and were considered to be zero.

Water budget evaporation

The average monthly evaporation of Lake Ontario for the period 1934 to 1953 computed by the water budget method was obtained by substituting the values of the normal change in levels, runoff, inflow minus outflow, and precipitation over the lake in Equation (1). The values so obtained are given in Column 7, Table 1 above. Note that the minimum average monthly evaporation occurred in June and the maximum average monthly evaporation occurred in October. The reason for this will be explained below. The total annual average evaporation for the period was 2.21 feet.

Mass Transfer Method

The velocity of a viscuous fluid at the zone of contact with a solid boundary must have the same velocity as the solid boundary. Thus, a free stream of air moving with the velocity U past a fixed solid boundary must have a velocity of zero at the boundary and a velocity of U some distance away. Where a velocity gradient is found, the turbulent motion leads to a transport of momentum across surfaces normal to the velocity gradient. A shearing stress is exerted on a surface across which a transfer of momentum takes place. The relatively thin layer where the velocity gradient is large is called the boundary layer. Not only is the character of the fluid flow, laminar or turbulent, important, but the nature of the boundary has an important function. The flow of wind over Lake Ontario was considered to be turbulent and the surface aerodynamically rough.

The general equation for shearing stress is:

$$\tau = A \frac{dU}{dz} \tag{2}$$

where

τ is the shearing stress.

A is the coefficient of turbulent interchange of momentum.

$\frac{dU}{dz}$ is the gradient of horizontal wind velocity.

The transfer of water vapor through the atmosphere was considered to occur in the same manner as the transfer of momentum. Thus:

$$E = -A \frac{dq}{dz} \tag{3}$$

where

E is the evaporation.

A is the coefficient of turbulent interchange of moisture.

$\frac{dq}{dz}$ is the gradient of moisture concentration.

The specific humidity is defined as

$$q = \frac{.622e}{Pa - .378e} \tag{4}$$

where

e is the vapor pressure of the air

Pa is the atmospheric pressure.

Since Pa is of the order of one hundred times e, there is little loss of accuracy by substituting

$$q = \frac{.622e}{Pa} \tag{5}$$

Since the coefficient of turbulent interchange of moisture was considered to be equal to the coefficient of turbulent interchange of momentum, the evaporation was considered to be a function of the friction velocity, U_*, and the difference between the vapor pressure of the air and the vapor pressure of saturated air at the water surface.

$$E = K\, U_* \Delta e \tag{6}$$

where U_* is the friction velocity and is equal to $\sqrt{\tau/\rho}$

Equations for the value of the shear stress of the wind over water under all conditions of temperature stability have been determined.(3) In using these equations, it was assumed that when the water surface is covered with waves different equations must be used than when there are no waves on the surface. There appeared to be a critical value of U_* for which the water surface becomes unstable, and waves form.

Excellent data obtained from water-loss investigations at Lake Hefner were used to correlate measured evaporation with that computed by Equation (6) above. The equation developed from the Lake Hefner data for evaporation in inches per day which was considered to be applicable to Lake Ontario was

$$E = .000733\, U_* \,(e_2 - e_o) \tag{7}$$

where

U_* is in centimeters/second.

e is in millibars and the vapor pressure of the air is measured at 2 meters elevation.

The correlation factor of Lake Hefner measured evaporation to that computed by the above formula was 0.90. The equation also appeared to give good results for the Lake Mead studies.

It is an extremely difficult task to obtain wind and temperature data for Lake Ontario. The friction velocity (U_*) is a function of the wind velocity over water at a given height and the atmospheric stability. The measure of atmospheric stability used was the difference between the air and water surface

temperatures ($T_A - T_W$). There are only two anemometers operating the year round near Lake Ontario where the wind velocities measured over land can be adequately reduced to over-lake wind velocities. Both of these anemometer locations, Toronto and Cobourg, are in Canada, and, unfortunately, the Cobourg anemometer was discontinued in 1952. The recorded wind speeds were first reduced to eight meter level wind speeds and then were increased to over-water winds in accordance with experience gained in relating vessel weather observations of Great Lakes anemometer-equipped vessels with land stations.(4) The factors by which the over-land wind speeds were increased depended upon the atmospheric stability; these are given in Table 5.

TABLE 5

Ratio of Over-Water to Over-Land Wind Speeds, $\frac{U_W}{U_L}$

$T_A - T_W$ in ° F		
$\leq -8°$	$\geq -7°$ to $\leq +7°$	$\geq +8°$
1.50	1.21	1.00

The vapor pressure differential (Δe) is a function of the temperatures of the air, the relative humidity of the air, and the water surface temperature. The air temperatures and the dew points were derived by arithmetically averaging the values recorded at Toronto, Trenton, and Oswego, for the period 1937 through 1952. The water surface temperatures were derived from the work of Freeman, Millar, and the U. S. Lake Survey, see Table 6.

The evaporation computed by the mass transfer equation, Equation 7, is given in Table 7.

Using the mass transfer method, the minimum average monthly evaporation occurred in May and June and the maximum average monthly evaporation occurred in September. The difference in vapor pressures in May and June

TABLE 6

Lake Ontario Water Surface Temperatures
Degrees Fahrenheit

Month	Freeman[1]	Millar[2]	U. S. Lake Survey[3]	Adjusted Temperatures
January	32	37	32	32
February	32	35	32	32
March	33	35	36	35
April	39	37	41	39
May	46	42	48	45
June	57	54	58	56
July	67	67	65	67
August	68	69	67	68
September	64	65	63	64
October	54	55	56	55
November	41	45	44	44
December	33	39	34	34

[1]"Regulation of the Great Lakes," report by John R. Freeman.

[2]"Surface Temperatures of the Great Lakes," F. Graham Millar, Journal of the Fisheries Research Board of Canada, December 1952.

[3]"Archives, U. S. Lake Survey, Detroit, Michigan

shows that there was no appreciable gradient of moisture concentration and, consequently, no net loss of water mass. In fact, in some years, there was a negative evaporation in the months of May and June.

Discussion

All data from known sources were used in this study. However, the available data were not of the highest order, consequently, the computed evaporation is subject to error. New installations must be established to collect more complete and accurate meteorological and hydrological data in the

TABLE 7

Evaporation in Feet Computed by Mass Transfer Equation

Month	Over Water Wind Velocity 8 Meter Elevation Centimeters/second	U* Centimeters/second	Δ e Millibars	E Foot/Month
January	832	43.4	.081	.23
February	806	42.7	.077	.19
March	636	34.1	.073	.16
April	588	36.0	.039	.09
May	398	26.9	.001	.00
June	338	22.4	.013	.02
July	374	21.7	.132	.18
August	392	21.6	.157	.22
September	455	22.8	.191	.27
October	479	23.8	.150	.23
November	605	31.5	.100	.20
December	818	44.1	.076	.21
Yearly Total				2.00

Great Lakes area if the most beneficial regulation plans are to be effected.

A comparison of the evaporation computed by the water budget and the mass transfer methods is given in Table 8.

The results compare rather favorably and the trend of evaporation is the same, although there are minor differences in the amount of net evaporation. The evaporation in the spring months has been shown to be negligible because there is no appreciable gradient of moisture concentration. The maximum evaporation occurs in the fall.

There is definite evidence that the precipitation falling over the surface of the Great Lakes is considerably less than the precipitation falling on the adjacent areas. The fact that the precipitation over Lake Ontario is some 20 per cent less than the precipitation recorded at land stations throughout the basin means that the values of evaporation are reduced in the same amount when computed by the water budget equation. As a result, the annual amount

TABLE 8

A Comparison of Computed Evaporation, in Feet

Month	Water Budget Method	Mass Transfer Method
January	.19	.23
February	.10	.19
March	.06	.16
April	.04	.09
May	.04	.00
June	.00	.02
July	.11	.18
August	.28	.22
September	.32	.27
October	.38	.23
November	.35	.20
December	.34	.21
Total	2.21	2.00

of net evaporation from Lake Ontario computed by both the water budget and the mass transfer methods is of the order of 2.0 feet.

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PROBLEMS CONCERNING USE OF LOW HEAD RADIAL GATES

Thomas J. Rhone,[1] A.M. ASCE

ABSTRACT

Three of the principal hydraulic features of low head radial gates are discussed. These are (1) a general discharge equation, (2) the effect of gate seat location on discharge capacity and pressure distribution along the spillway surface, and (3) side and bottom seals.

INTRODUCTION

The radial-type gate originated in France about 100 years ago.(1) The earliest recorded use was by the French Engineer Poiree on the Seine River in 1853. In the 1860's, another French Engineer, Mongel Bey, used a cast-iron, radial-type gate in the Delta Barrage on the Rosetta Branch of the Nile River. The Poiree gates were 28.7 feet wide by 3.3 feet high; the Delta Barrage gates were 16.4 feet wide by 16.7 feet high with the concave face turned toward the reservoir.

The radial-type gate was patented in the United States in 1886. Since then, this type of gate has become widely used and has increased in size. Three 114 feet wide by 26 feet high gates are used on Horseshoe Dam Spillway near Phoenix, Arizona, and it is proposed to use four 40-foot wide by 52.5-foot high radial gates for reservoir control at the new Glen Canyon Dam.

Radial or Tainter gates are probably the most widely used crest control gates. They are particularly well adapted to crest control because of their simplicity of design, construction, and installation. The overall economy and efficiency of this type of gate result from the radial bearing. The thrust from the waterload is carried to two trunnion bearings where it offers little resistance to the operation of the gate. This action can be compared to the hub of a

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1. Hydraulic Research Engr., Division of Eng. Lab., Bureau of Reclamation, Denver, Colo.

wheel where all of the force on the rim is transmitted to the center. In lifting the gate, there is some force to be overcome at the trunnion, and the side seals may offer considerable resistance in the form of friction. However, the opening of a radial gate requires less hoist capacity than a slide gate with face bearing. In addition, radial gates are more adaptable for automatic control apparatus.

Another desirable feature of a radial gate is that it needs no gate slots. High head flow past indented slots has in some installations produced cavitation damage to the pier and spillway surfaces. One instance where gate slots were the apparent cause of cavitation to spillway and pier side walls is shown in Figure 1; several other cases have been reported in technical literature.[2] Preventive action has included the use of gate followers to fill the slots at small gate openings, and the use of offsets in the pier side walls downstream from the slots.

Despite the apparent simplicity and wide use of radial gates, very little seems to be known about their operating characteristics. Specifically, there has been very little information published on the discharge capacity, the effect of gate location on pressure distribution on the spillway face, downpull produced by flow under the gate, and the type of seals to be used, and other hydraulic problems.

The purpose of this paper is to discuss some of the hydraulic features of radial gates, including the side and bottom seals; to present hydraulic data on some representative gate installations; and to create interest in the remaining problems, with the hope that additional material will be presented by the readers of this paper. Further, it is hoped that the tests described will stimulate hydraulic research on radial gates by those in a position to do hydraulic model or prototype investigations.

Material in this paper has been limited to the so-called low head radial gates used to control flow over spillways. Top seal radial gates, where the normal operating head greatly exceeds the gate height, and the small gates used to control canal flow, are not covered.

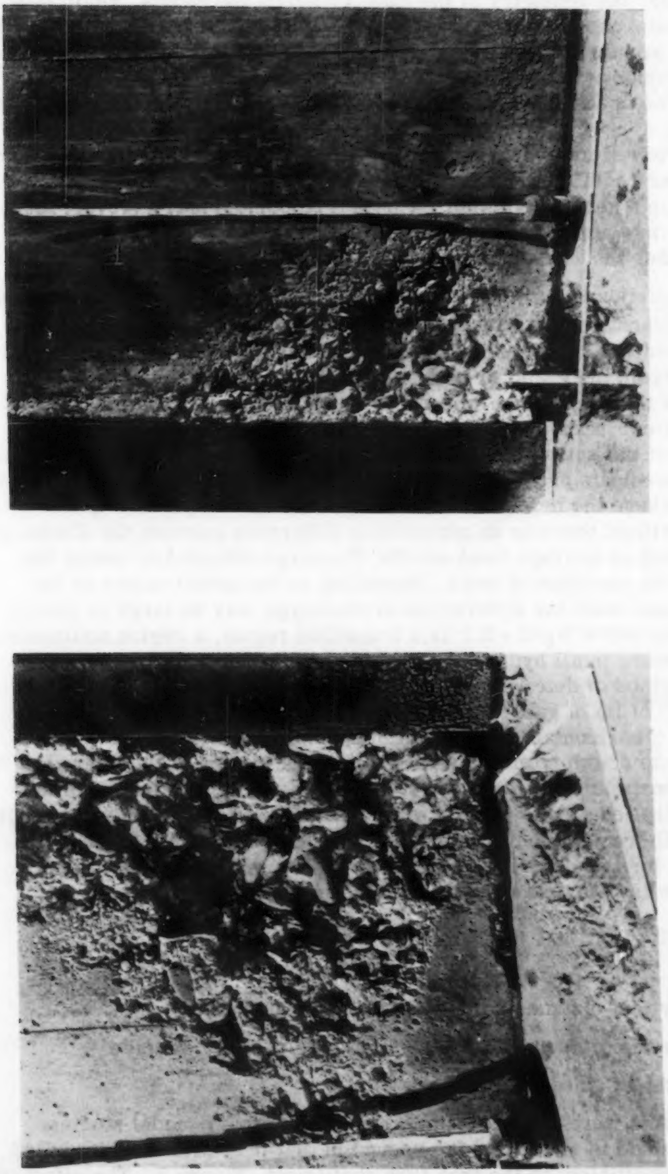
Discharge Characteristics

Discharge Determination

Although the radial gate has been used for regulating flow for over a hundred years, a general method for predicting exact flow quantities has not been derived except for one type of installation. When the gate is used in a flat-bottom, rectangular section and the discharging jet is supported by the flat floor, the flow characteristics are known both from mathematical and experimental analyses. The discharge coefficients derived from these studies have been confirmed by several experimenters and are accurate for their specific purpose.[3,4]

Many specific discharge determinations are on record; usually, when a spillway has been model-tested, calibration curves for both free and gate-controlled flow have been obtained. From an analysis of these calibrations, the free flow discharge coefficients for almost any shape of overflow section can be determined.[5] However, a general equation which will provide the exact discharges for radial-gate-controlled flow is still unpublished.

The lack of a general equation can probably be explained by the many variables that affect the flow pattern. Some of these are in the spillway approach



CAVITATION DAMAGE DOWNSTREAM
FROM GATE SLOTS
PARKER DAM SPILLWAY

FIGURE 1

and include channel width and depth. Other variables include the shape of the spillway profile; the geometry of the gate, including the radius, width, height, trunnion location, and the location of the gate seat; the location, spacing, and type of piers; and the method of determining gate opening and reservoir elevation. The combinations of these variables are almost countless, and the effect of any one of them is difficult to determine from a limited number of investigations.

A brief review of four methods which have been used to determine the flow quantities under radial gates indicates the different approaches used in attempts to obtain a general solution. It should be pointed out, however, that each provides only approximate discharges; if an accurate determination is necessary, either model studies or field calibrations should be made.

The first equation is $Q = 2/3\ CL\ \sqrt{2g}\left[h_1^{3/2} - h_2^{3/2}\right]$ where "C" is the coefficient of discharge listed as a function of gate opening and reservoir head. The definition of the symbols and suggested values for "C" are shown in Figure 2. From the graph, it is apparent that for h_1/d values less than about 2.2 there is a wide range where "C" can vary by as much as 20 percent. The coefficient curve shown as the heavy line was derived from hydraulic model studies of the spillways listed on Figure 2. The different crest shapes and gate arrangements probably account for some of the spread. However, it is known that where the head on an orifice is small in comparison to the height of the orifice, there is an appreciable difference between the discharge obtained by using an average head and the discharge obtained by taking into consideration the variation in head. Depending on the actual values of the orifice height and head, the differences in discharge may be large or small. Thus, the region below $h_1/d = 2.2$ is a transition region, a region notoriously difficult to evaluate in all hydraulic flow problems.

A second method of determining discharge quantities takes into consideration the angle θ of lip of gate to horizontal and the angle Δ described by the intersection of a horizontal line through the pivotal point of the gate and the radius of the gate drawn from the same point to the center of the gate when it is in a closed position, Figures 3 and 4. The value of the coefficient of discharge for controlled flow is related to the known coefficient of discharge for free flow for different values of θ and ratios of gate openings d to depth of flow h_1. Plots for two different values of θ are shown in Figures 3 and 4. The rate of flow is then obtained by applying the formula

$$Q = CL\left[h_1^{3/2} - h_2^{3/2}\right]$$

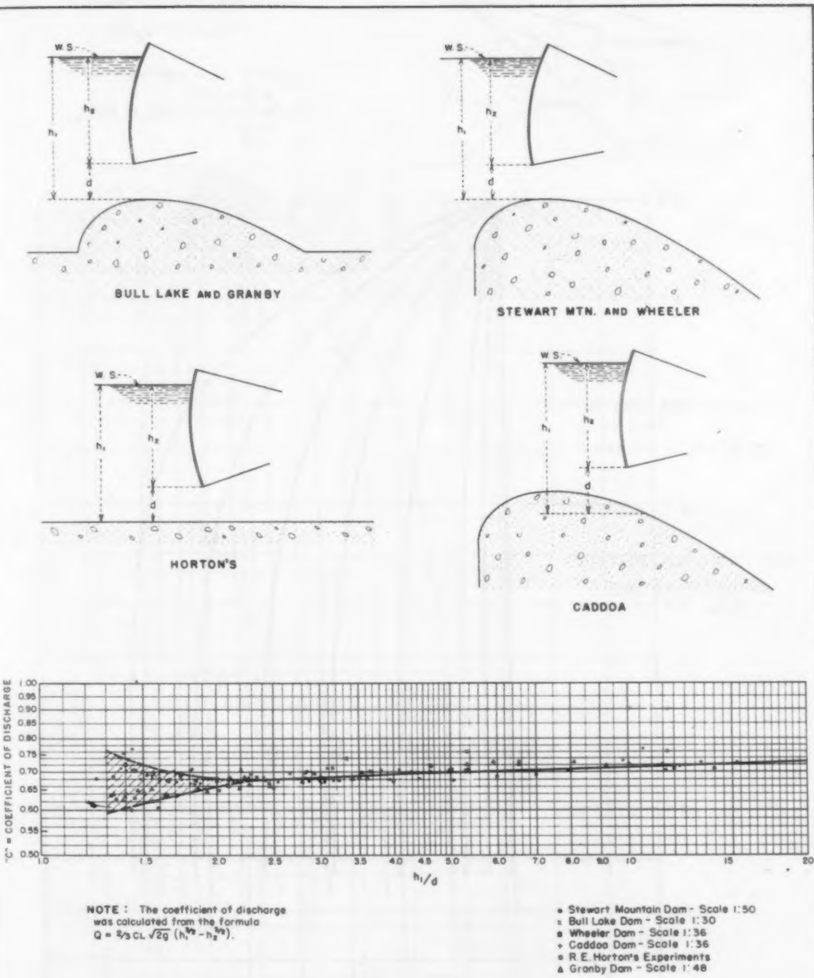
where h_1 = vertical distance from crest to water surface

h_2 = vertical distance from bottom of gate to water surface

L = width of gate

The coefficients were derived from the results of many model studies.

The third method is similar to the second in that it relates the controlled flow discharge coefficient to the free flow coefficient, Figure 5. The effect of gate radius, trunnion location, and gate opening is also recognized as having an effect on the coefficient and is evaluated by the parameter $\dfrac{1}{\sin\theta}$. The values for "C" were obtained from analyses of calibrations of model spillways.



COEFFICIENT OF DISCHARGE
RADIAL GATES

FIGURE 2

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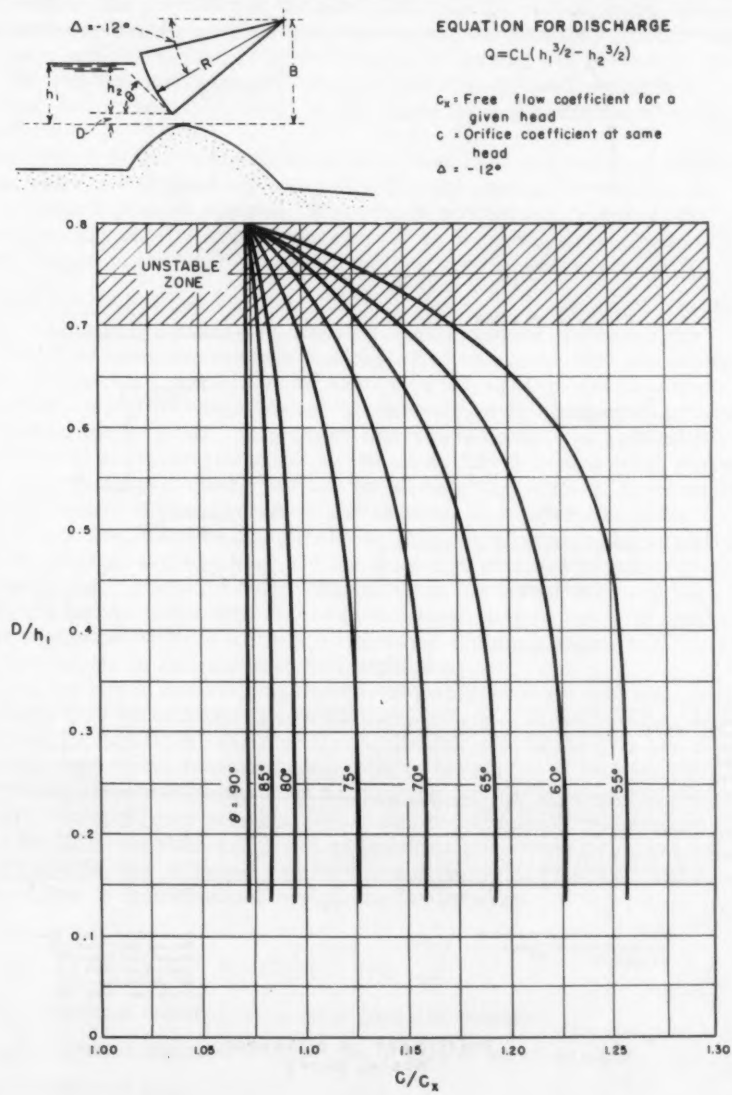
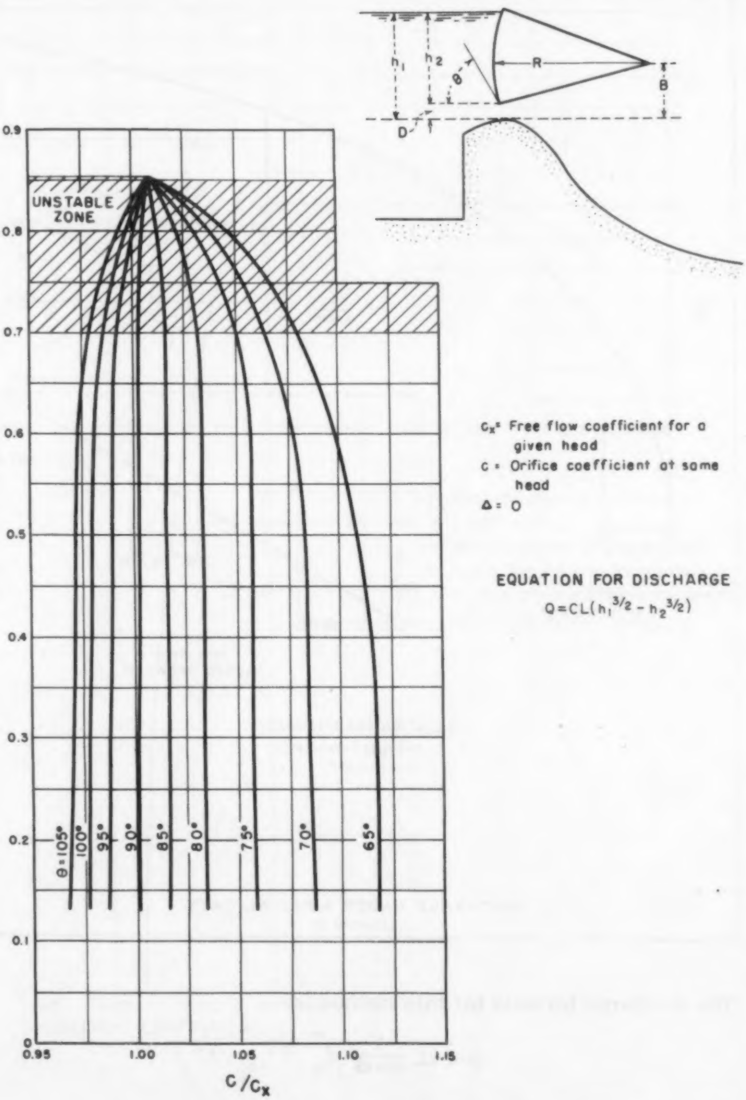
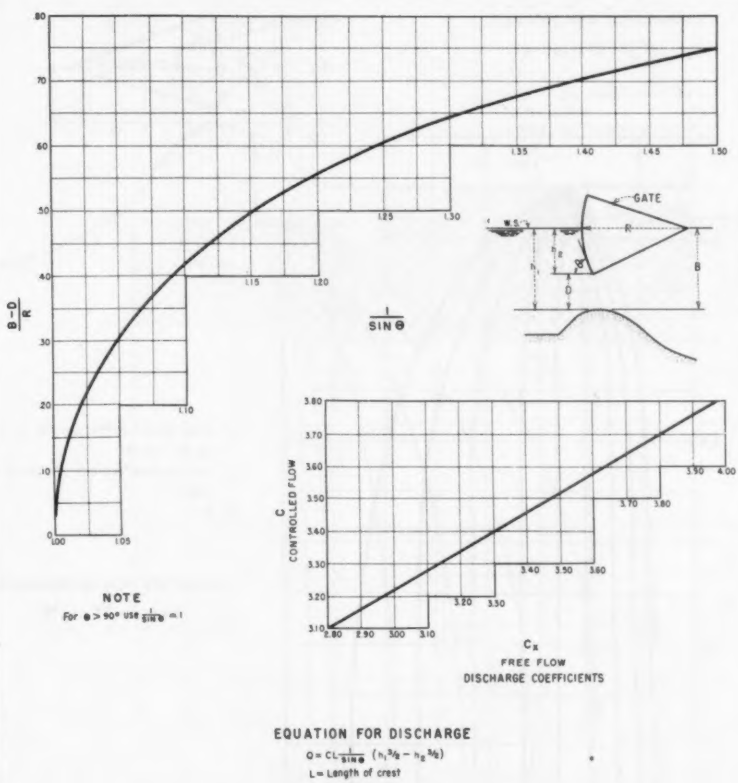


DISCHARGE COEFFICIENTS FOR RADIAL GATES
FIGURE 3



DISCHARGE COEFFICIENTS FOR RADIAL GATES
FIGURE 4



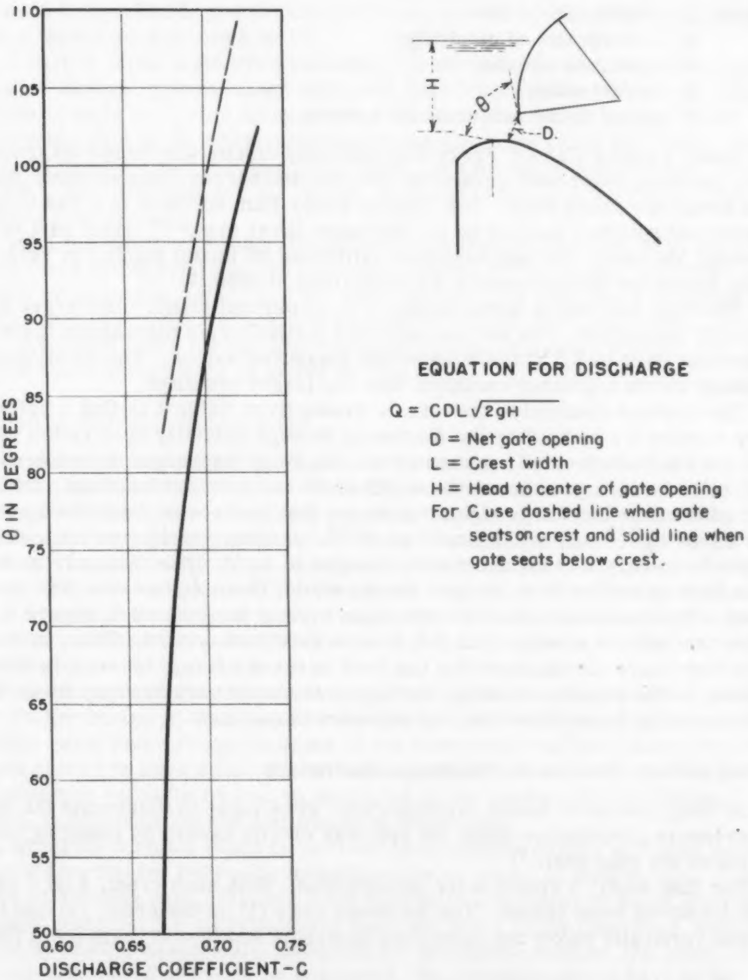
DISCHARGE UNDER A RADIAL GATE
FIGURE 5

The discharge formula for this method is:

$$Q = CL \frac{1}{\sin\theta} \left[h_1^{3/2} - h_2^{3/2} \right]$$

Values for CL and $1/\sin\theta$ are given in Figure 5.

The fourth method, Figure 6, takes into consideration the crest shape, location of gate seat, the angle θ formed by the tangent to the gate lip and the tangent to the crest curve at the nearest point of the crest curve, and the net gate opening which is the shortest distance from the gate lip to the crest curve. From the geometry of the gate the angle θ is determined and from the curves in Figure 6, a coefficient of discharge is determined. These values



DISCHARGE UNDER A RADIAL GATE

FIGURE 6

for "C" were also obtained from model investigations. The discharge is then computed from the formula:

$$Q = CDL \sqrt{2gH}$$

where Q = discharge in cfs
C = coefficient of discharge
D = net gate opening
L = crest width
H = head to the center of the opening

Table 1 shows Canyon Ferry Dam Spillway discharges computed from the four methods described, compared with the discharges obtained from model and prototype calibration. The Canyon Ferry Dam Spillway is a radial-gate-controlled spillway located on the Missouri River about 17 miles east of Helena, Montana. The spillway was calibrated by model studies in 1953, and field discharge measurements were obtained in 1956.(6)

The first and fourth methods give 5 to 11 percent higher discharges than actually measured. The second and third methods give discharges from 2 percent over to 7.5 percent under the measured values. The small gate opening shows a greater variation than the larger openings.

The obvious conclusion that can be drawn from Table 1 is that a satisfactory method for computing the discharge through partially open radial gates has not yet been derived. A method of computing discharges is required which will be in better agreement with model tests and field calibrations. Examination of the four discharge equations shows that in three of them the discharge varies as $h_1^{3/2}$ and in the fourth as $H^{1/2}$. A more nearly correct method might be to vary this exponent with changes in h_1/d. The change from free weir flow to orifice flow, or vice versa, would, thereby, be taken into account. The transition range occurs when h_1/d is less than 2.2, Figure 2. When the ratio is greater than 2.2, true orifice flow occurs. Thus, in the free flow range the exponent for the head in the discharge formula is three-halves; in the transition range, the exponent should vary between three-halves and one-half; for orifice flow, the exponent is one-half.

Effect of Gate Position on Discharge Coefficient

In 1953, hydraulic model investigations were made to determine the effect on pressure distribution along the spillway profile caused by changing the location of the gate seat.(7)

For this study, 5 crests were investigated. With each crest, 4 or 5 gate seat locations were tested. The locations were (1) on the crest, (2) and (3) 6 inches vertically below the crest both upstream and downstream from the

Table 1

Gate opening*	Head*	Proto Q (cfs)	Model Q (cfs)	Computed Q (cfs) 1	2	3	4
3 ft	32.32	4,400	4,450	4,870	4,110	4,060	4,880
6 ft	32.00	8,450	8,400	9,240	8,370	8,270	8,940
10 ft	32.45	13,800	13,750	14,720	14,070	13,980	14,510

*Gate openings and heads are measured vertically above gate seat.

crest axis, (4) 12 inches below the crest on the downstream side, and (5) the design gate seat location when it did not correspond to one of the other four.

The five crests and the model scale ratio at which each was studied are (1) Ross Dam 1:24.14, (2) Altus Dam 1:20.70, (3) American Falls Dam 1:18.35, (4) Bhakra Dam 1:50.56, and (5) Canyon Ferry Dam 1:42.92. The crest profiles are shown on Figures 8 to 12.

As a part of these tests, data necessary to compute the coefficient of discharge for several gate openings with each gate location were obtained. The coefficients were computed using the first method described. It was found that when the gate seats either upstream from the crest or on the crest, the coefficient of discharge is higher than when the gate seats downstream from the crest. This was true for all crests; in some cases, the difference was as much as 15 percent.

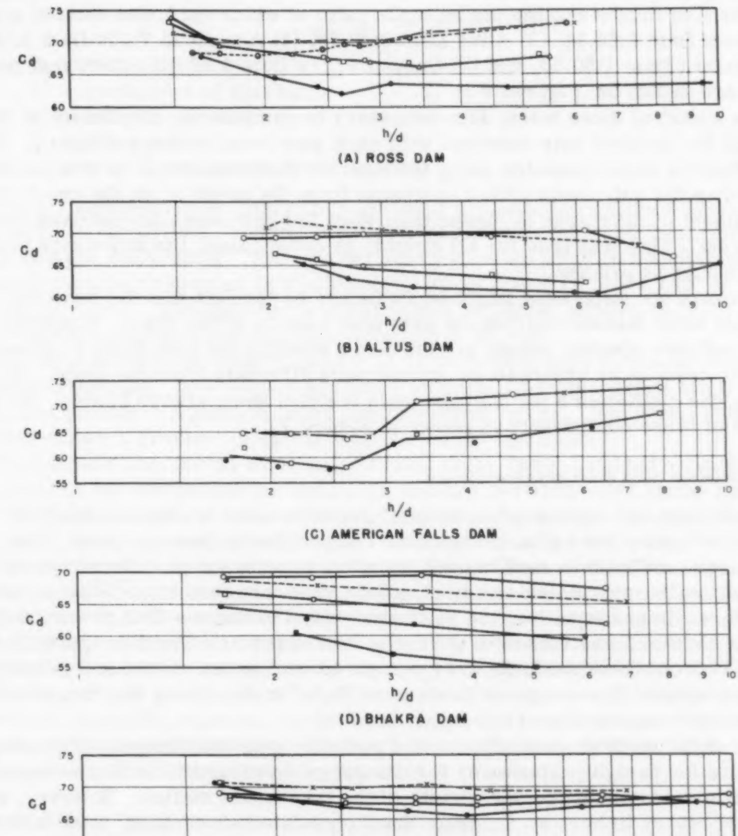
Some of the difference might be explained by the fact that the head and gate opening were measured from the gate seat instead of the crest. However, the head and gate opening values are the same whether the gate seats 6 inches below the crest in an upstream or downstream direction from the crest. In all cases, the coefficients for the upstream position were always higher. An example of these coefficients is shown in Figure 7.

Conclusions and Recommendations

Four different discharge equations have been used to compute the flow quantities under the radial gates of the Canyon Ferry Dam spillway. The discharge coefficients used in each equation were based on different model studies; each equation placed emphasis on different geometric features of the structure. In one equation, the head was raised to the one-half power; in the others, the head was raised to the three-halves power. The flow quantities obtained from these methods were compared with actual model and prototype measurements; the computed flows were found to vary from the measured quantities by as much as 11 percent.

All of the methods described would probably give discharges sufficiently accurate for design purposes or for discharge determinations where water has little value and a rough estimate of the flow would suffice. However, when it is necessary to have an accurate discharge determination for flood routing or to measure valuable irrigation water, as in the western United States, either a model or prototype calibration is necessary.

An accurate formula could be derived by combining theory and data analysis from coordinated hydraulic investigations on model or prototypes. The amount of existing data available from hydraulic laboratories might provide sufficient material for a preliminary analysis and if these data were supplemented by models specifically designed for discharge studies, the effect of each variable could probably be evaluated. Such models would have to be sufficiently large that changes in the geometry of the gates, piers, and overfall section would make measurable changes in the flow. The crest length should be sufficiently long to allow installation of several piers, and provision should be made to test several crest shapes, pier shapes, and gate sizes. It would be desirable to incorporate an adjustable weir in the model headbay, which would maintain a constant reservoir level to facilitate the flow measurement in the transition range between free flow and orifice flow. Standard measuring devices for determining gate openings, discharge quantities, and heads on the crest would be adequate. With such a model, the effect of any of



EXPLANATION OF SYMBOLS

- C_d Coefficient of discharge in $Q = 2/3\, C_d\, L\,(H^{3/2} - h^{3/2})\sqrt{2g}$
- Q Discharge in cfs
- L Spillway width
- H Vertical distance from gate seat to reservoir elevation
- h Vertical distance from gate lip to reservoir elevation
- d Vertical distance from gate seat to gate lip
- ■ Gate seat 2.43 ft. below crest, on downstream side
- ● Gate seat 12 inches below crest, on downstream side
- □ Gate seat 6 inches below crest, on downstream side
- △ Gate seat 4 inches below crest, on downstream side
- × Gate seat on crest
- ○ Gate seat 6 inches below crest, on upstream side

EFFECT OF GATE POSITION ON DISCHARGE COEFFICIENT
FIGURE 7

the variables such as approach depth, pier shape, or trunnion location could be evaluated.

Pressure Characteristics

A standard spillway crest is usually designed to conform to the shape of the lower nappe downstream from a sharp-crested weir. This profile is called the datum shape.(5) When gates are to be used on the spillway, the same crest shape is used from the upstream face to the gate seat. Downstream from the gate seat, the crest profile is sometimes made to correspond to the theoretical trajectory of a jet flowing from a 1-foot high rectangular orifice at the design head. For the same design head, the crest designed for free flow will be steeper than a crest designed for a 1-foot gate opening.

Several factors other than hydraulic phenomena are usually considered when determining the location of the gate seat. If the gate seats on the crest, the theoretical crest curve will be comparatively flat and the spillway will contain considerably more concrete than is usually needed for stability. If the gate seats below the crest, the profile will be steeper, but a higher gate is necessary to retain the same storage pool. Structural and architectural considerations sometimes dictate the gate seat location. If a bridge spans the spillway, it might be necessary to place the gate downstream from the crest to provide space for the gate lifting mechanism or to provide clearance when the gate is raised. The gate might have to be located downstream to insure that the trunnion will be above the water surface.

Many hydraulic designers believe that when the gate is seated on the crest there is a tendency for the jet to spring free of the crest at high reservoir elevations and small gate openings; this may result in subatmospheric pressures and possible cavitation damage to the spillway. The designers also believe that placing the gate seat downstream from the crest results in the flow being directed downward; the tendency of the jet to spring free of the crest is lessened. However, observations of prototype spillways indicate that cavitation damage to the spillway surface occurs downstream from the region where the control gates have any effect on the flow. No damage to spillway crests in the vicinity of the gates has been found which can be attributed to the crest shape and gate seat location.

A realistic solution to these problems and conflicting requirements is needed. The following discussion may help to clarify the problems.

Effect of Gate Location on Pressure Distribution Along a Spillway Profile

The laboratory investigations used to determine the effect of gate location on discharge coefficients were also used to determine the effect of the gate position on the pressure distribution along the spillway surface.(7) For the pressure tests, 10 piezometers were installed along the crest profile. One piezometer was placed at each tested gate seat and the others were equally spaced downstream along the spillway profile. The five crests used in the investigation provided a wide range of steepness and gate seat locations.

For convenience, each of the crests will be discussed separately giving a description of the crest and the results of the tests. The crest profiles, datum shapes, piezometer locations, and pressure profiles for representative gate openings are shown on Figures 8, 9, 10, 11, and 12.

Ross Dam, Figure 8

The design gate seat is 4 inches below the crest. The datum shape is slightly steeper than the design shape. A comparison of the design shape and datum shape indicates that the overfall section from the gate seat downstream was designed from the theoretical trajectory of a jet from an orifice. From the reasoning previously outlined, pressures along the profile should increase when the gate seat is downstream from the design location and reduce when the gate seat is upstream.

The pressure profiles on Figure 8 show that for the same gate openings and heads, the pressures for all gate seat locations downstream from the design gate seat are practically the same. However, when the gate is seated upstream from the crest, the piezometers from the crest down to the second piezometer below the design gate seat showed pressures lower than were obtained for the other gate seat locations. Another trend which may be observed from the pressure profiles is that the zone of subatmospheric pressures moved upstream as the gate seat moved upstream. However, for all gate positions, the downstream piezometer (No. 9) showed pressures near atmospheric.

Altus Dam, Figure 9

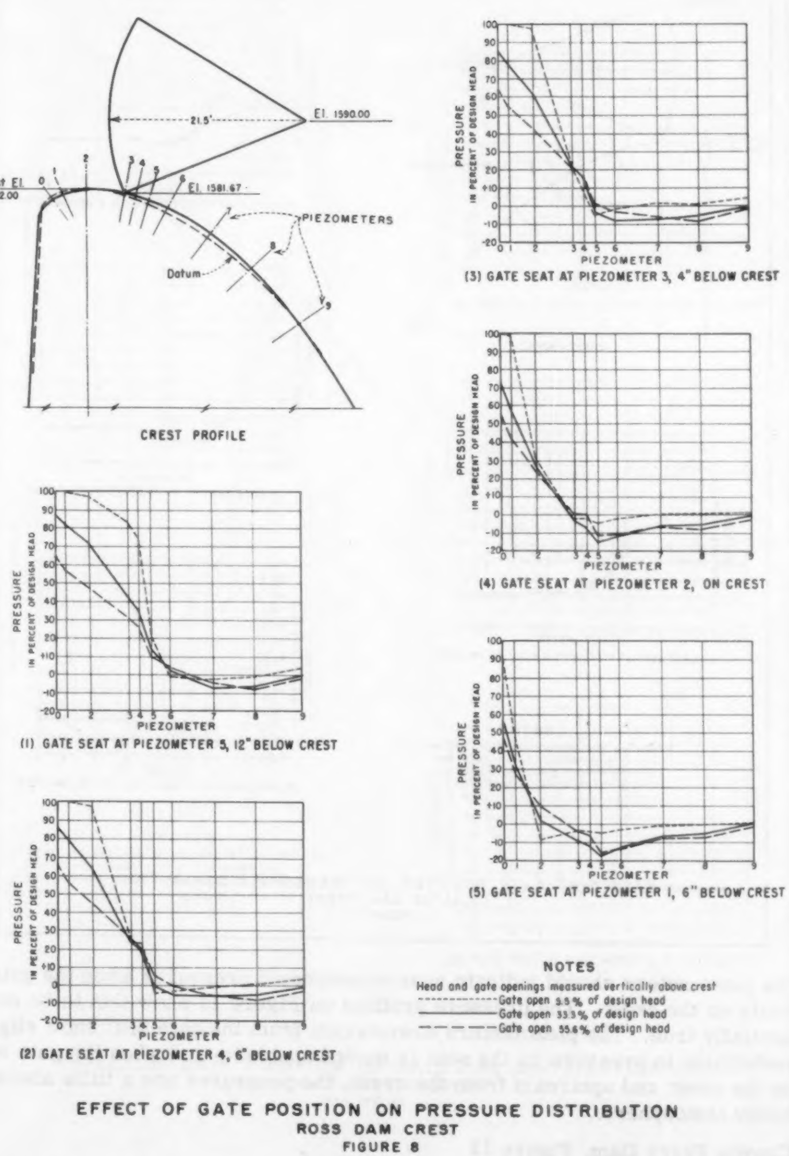
The design gate seat is on the crest. The datum shape is the same as the design shape, indicating that the profile was obtained from the lower nappe shape of a sheet of water passing over a sharp-crested weir. According to reasoning, the pressures along the profile should have the highest values when the gate seat is 12 inches below the crest and should become smaller as the seat is moved upstream. The pressure profiles in Figure 9 show this to be true in that for both downstream gate seat locations the piezometers downstream from the gate seat indicate nearly atmospheric pressures for all gate openings. Also, when the gate seats upstream from the crest, the piezometers from the gate seat to Piezometer No. 6 indicate subatmospheric pressures. The piezometers downstream from No. 6 all show near atmospheric pressures. Generally speaking, Piezometers No. 6 through 9 showed nearly atmospheric pressures for all gate seat locations at all gate openings. Piezometers No. 2 through 5 were affected by the gate seat location and the pressure was reduced at each piezometer as the gate seat was moved upstream.

American Falls Dam, Figure 10

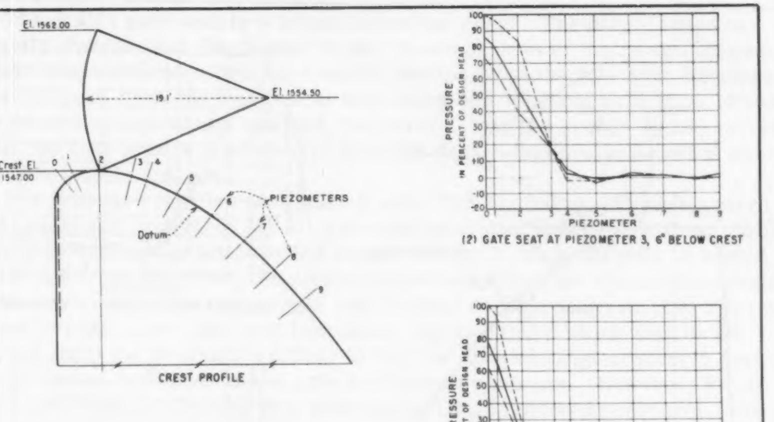
The design gate seat is on the crest. The datum shape is considerably steeper than the design shape, indicating that the profile from the crest downstream was obtained from the theoretical trajectory of a jet flowing from an orifice. The pressures should be near atmospheric for small gate openings when the gate is at the design location, for downstream gate seat locations the pressures should be higher, and for upstream locations the pressure should be lower. The pressure profiles on Figure 10 indicate that performance agrees with the reasoning. The only pressures significantly subatmospheric were those at the crest piezometer when the gate seat was upstream from the crest.

Bhakra Dam, Figure 11

The design gate seat is 2.43 feet below the crest on the downstream side. The datum shape is steeper than design shape. For this crest, the highest pressures should occur with the gate at the design location and they should become successively lower as the seat location is moved upstream. Also,



EFFECT OF GATE POSITION ON PRESSURE DISTRIBUTION
ROSS DAM CREST
FIGURE 8

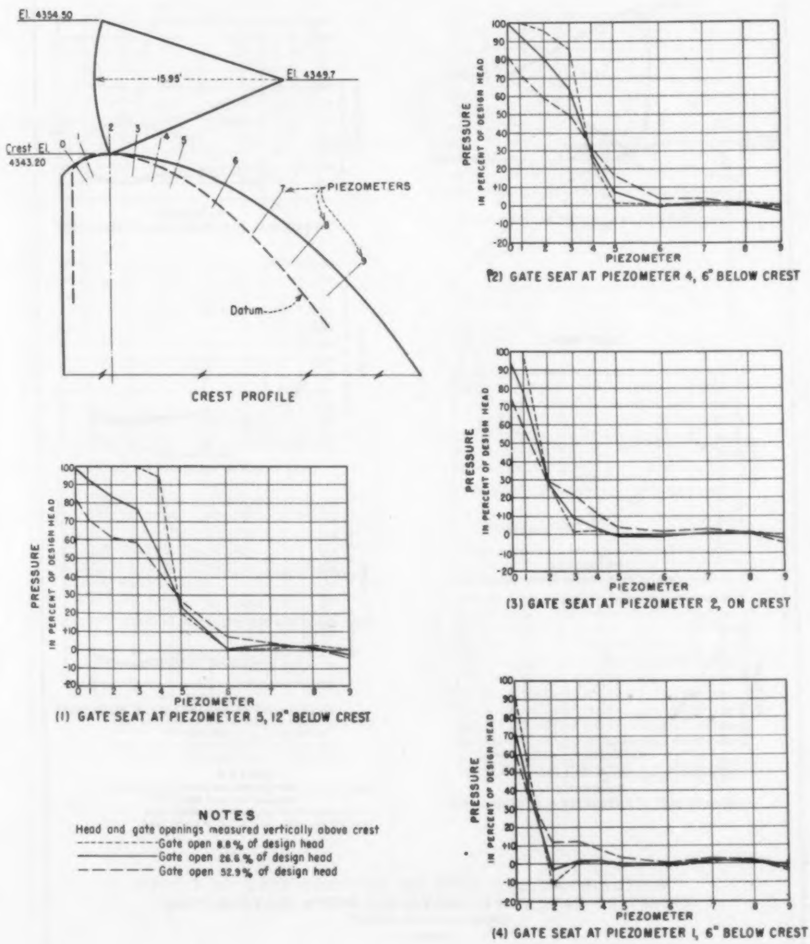


EFFECT OF GATE POSITION ON PRESSURE DISTRIBUTION
ALTUS DAM CREST
FIGURE 9

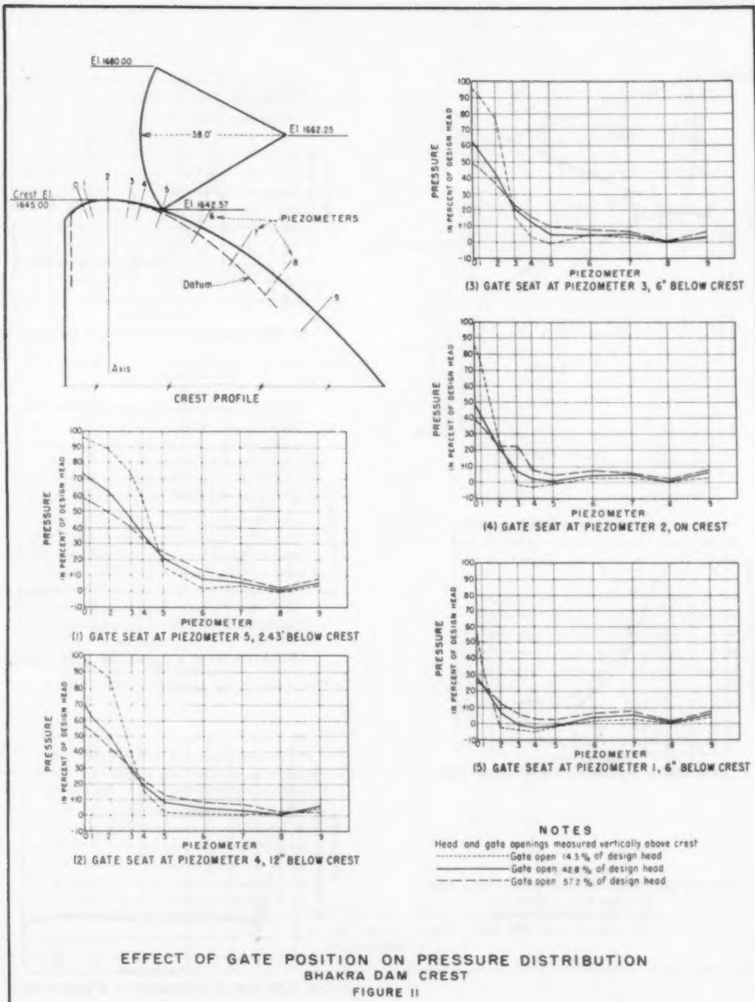
the piezometers should indicate near atmospheric pressures when the gate seats on the crest. The pressure profiles on Figure 11 show this to be essentially true. The piezometers downstream from the gate seat show slight reductions in pressure as the seat is moved upstream and when the gate seats on the crest and upstream from the crest, the pressures are a little above and below atmospheric.

Canyon Ferry Dam, Figure 12

The design gate seat is 6 inches below the crest. The datum shape is steeper than the design shape. The relative shape of the two profiles indicates that the profile downstream from the design gate seat was derived from the theoretical trajectory of a jet flowing from an orifice. Therefore, the

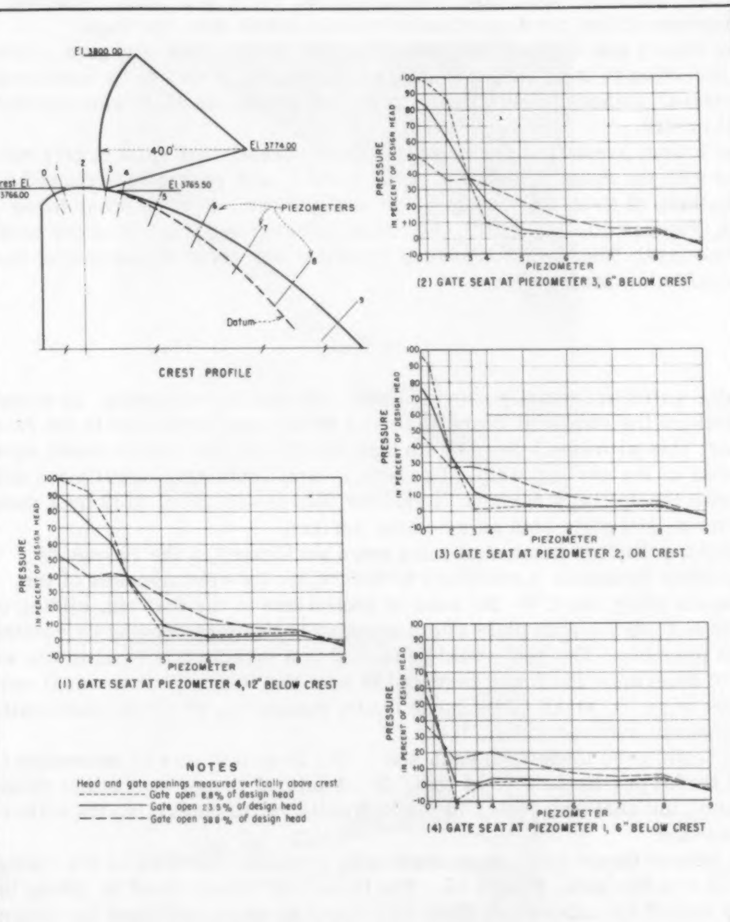


EFFECT OF GATE POSITION ON PRESSURE DISTRIBUTION
AMERICAN FALLS DAM CREST
FIGURE 10



EFFECT OF GATE POSITION ON PRESSURE DISTRIBUTION
BHAKRA DAM CREST
FIGURE 11

pressures downstream from the gate should be near atmospheric when the crest is at the design location, higher when the gate seat is downstream and lower as the gate seat is moved upstream. The pressure profiles on Figure 12 indicate that the pressures increase when the gate seat is downstream from the design seat, and decrease when the gate seat is upstream. However, for gate openings of 8.8 and 23.5 percent of the design head, the pressures



EFFECT OF GATE POSITION ON PRESSURE DISTRIBUTION
CANYON FERRY DAM CREST
FIGURE 12

downstream from a gate seat are practically the same for any gate seat location.

Conclusions

The pressure tests show that the gate seat location has a minor effect on the profile pressures when the gate seat is on or downstream from the crest.

When the gate seats upstream from the crest, there is a sharp reduction in the pressure at the crest, particularly at the small gate openings.

The theory that a gate seat downstream from the crest will give a downward direction to the flow passing beneath the gate seems to be confirmed by these tests. Although the increase in pressure was small, it was noticeable for all crests.

The lowest pressures occurred on the two crests that most nearly corresponded with the datum shapes, Figures 8 and 9, and generally occurred for a gate opening of about 33 percent of the design head. For the other three crests, Figures 10, 11, and 12, the lowest pressures occurred at the smallest gate openings. The lowest pressure recorded was about 20 percent of the design head below atmospheric.

Gate Seals

Radial gates are usually provided with side and bottom seals. In some installations the side seal bears against a steel plate embedded in the face of the pier; this provides a smooth surface for the contact and prevents excessive wear of the sealing strip. The bottom seal contacts a metal plate set flush with the spillway surface; a shallow depression about 3/32 by 3 inches in the metal sill plate acts as a seating surface.

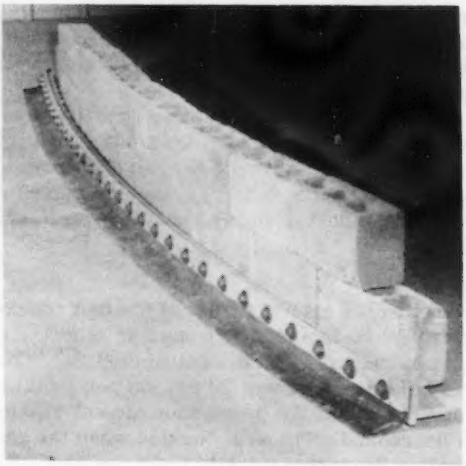
In 1952, a limited number of tests were performed in the Bureau of Reclamation Hydraulic Laboratory to determine the effectiveness of the gates seals being used.(8) Because of limitations in the test rig, sliding of the side seal on the wall plate corresponding to the gate opening or closing was not possible. The seal could be moved in a direction normal to the wall plate, to determine the force required to seat the seal. When the seal seated, leakages were too small to be conveniently measured, so visual observations of the leakage sufficed for the tests.

Two tests were made with each seal. The first test was to determine the lateral thrust per lineal foot of seal. If the lateral thrust was within reasonable limits, the seal was tested in the hydraulic rig to determine its action under pressure.

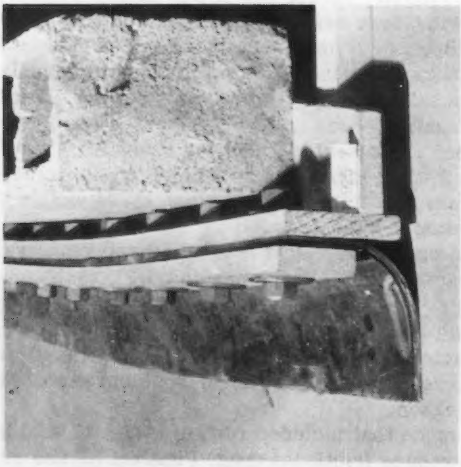
The lateral thrust tests were made with a wooden mock-up of the clamp plates of a radial gate, Figure 13. The thrust was determined by lifting the seal up and off the laboratory floor with a spring scale and steel bar placed under and perpendicular to the seal axis. The seal was lifted at a point midway between two pieces of paper placed 12 inches apart on the floor under the seal. When the paper could be slipped free, the thrust load on the floor was considered relieved and the spring scale load recorded. The frictional drag was computed from this thrust value.

Belt-type seal

One proposed side seal was made from fabric-reinforced rubber belting. A seal 8 inches wide and 15 feet long was assembled in the wooden mock-up for lateral thrust tests. The lateral thrust was determined for various seal positions. Zero seal position occurs when the space between the end of the gate and the wall plate is three-quarters of an inch. The tests showed that at the zero seal position, the lateral thrust was 16 pounds per lineal foot of seal. This thrust was high; and with the addition of a waterload at a 40-foot head, the frictional drag would be 75 pounds per inch of seal. In view of the high drag, no hydraulic tests were performed.



(A) General view of the 3/8 x 8 inch x 15 foot long belting seal bolted to a 40-foot radius.



(B) Close-up view of the above assembly.

FIGURE 13

RADIAL GATE SEAL TESTS

Wooden Mock-up of Rubber Belting Side Seal without Brass Shoes

Belt-type seal with brass shoes

It was decided to investigate the same seal with brass shoes bolted to the belting to reduce the frictional drag, Figure 14A. The lateral thrust at zero seal position for this seal was 158 pounds per lineal foot; but because of the smaller coefficient of friction with the brass shoe, the frictional resistance at a 40-foot head was 34 pounds per inch of seal.

Hydraulic tests showed that when the seal was moved away from the wall plate, the seal shoe slid downstream and remained in flat contact with the wall plate. However, when the gate was moved nearer to the wall plate, the brass shoe did not slide upstream on the wall plate, and the rubber belting buckled. Since this action was independent of the reservoir head, the seal was considered unacceptable.

Molded-type belt seal

As a means of reducing the lateral thrust of the belt type of seal, it was proposed that the belt seal be molded to an angular shape, Figure 14B. The lateral thrust of this seal was negligible at zero seal gap. The frictional drag on the wall plate was computed at about 20 pounds per lineal inch.

Hydraulic tests indicated that the brass shoe did not slip upstream freely as the seal gap was decreased. The seal buckled when the gap was decreased 1/4-inch, and the sealing lip raised slightly from the seat. It was concluded that the seal operation was only fair.

Angle seals

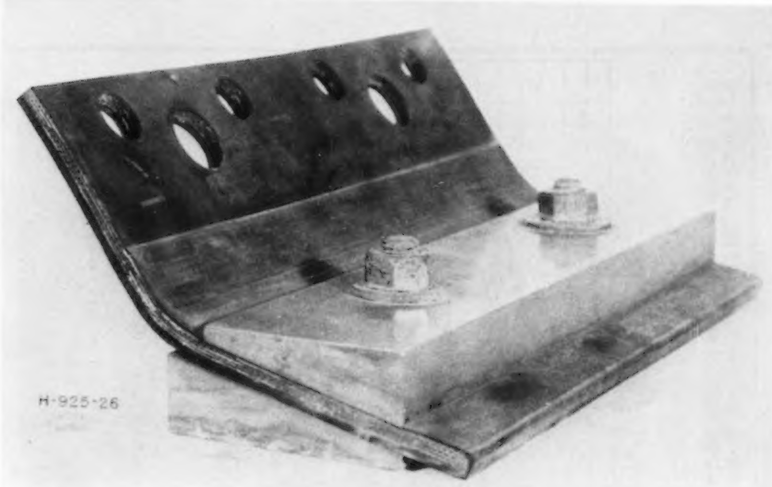
Two types of angle seals were investigated. One was an all-rubber seal, and the second was similar, except that it was fabric-reinforced, Figure 15. The lateral thrust with these seals would depend on how they were mounted on the gate sides. Because of this, tests were performed to determine at what head these seals would seal and unseal for various seal-to-side wall spacing. In addition, identical tests were made for the all-rubber seal with a brass bar on the sealing edge.

For the all-rubber angle seal, without the brass bar, the water head required for sealing at a 1/2-inch gap was 22 feet, while unsealing occurred at an 8-foot head, Figure 16. With the brass bar, these values were 23 and 7 feet. The fabric-reinforced seal sealed at a 19-foot head and unsealed at a 6-foot head. The computed frictional drag for these seals was about 37 pounds per lineal inch without a brass bar and 6.5 pounds per lineal inch with a brass bar.

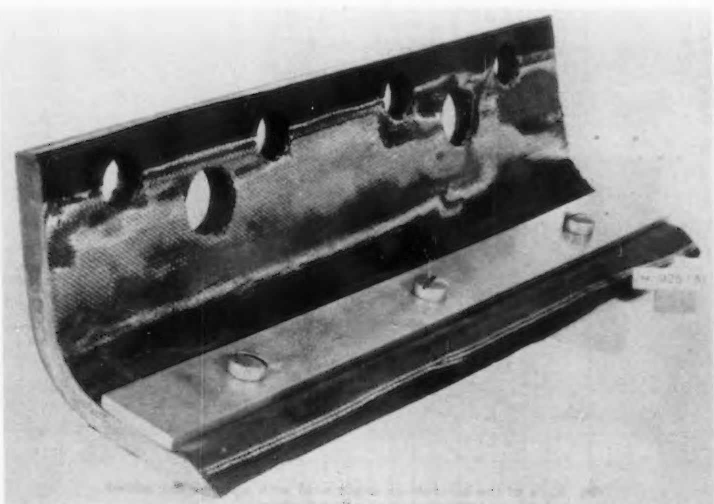
Although the angle seal with the brass bar appeared to have better sealing and unsealing characteristics, as well as a lower frictional drag, it allowed slightly greater leakage than the all-rubber seal and probably would be less desirable for this reason.

Two seals were made that included rubber-to-metal sealing surfaces and a brass member to reduce frictional drag, Figure 17. The seal with the rubber flaps (Figure 17A) was generally unsatisfactory, since the head required to obtain a water seal at normal gaps was higher than normally found for spillway gates.

The second seal, shown on Figure 17B, incorporated a more stabilized sealing lip. At a 1/2-inch gap, this seal sealed at a 20-foot head and unsealed at a 4-foot head, Figure 18. The frictional drag with this type seal at a 40-foot head and at zero gap is approximately 11 pounds per lineal inch.



(A) View of the rubber belting seal with brass shoes.

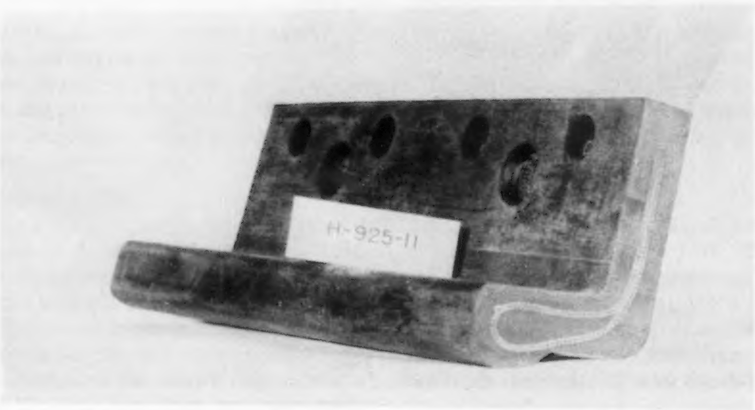


(B) View of the belt type, molded angle seal, with brass shoe, cut from a fabric reinforced angle seal shown on Figure 15A.

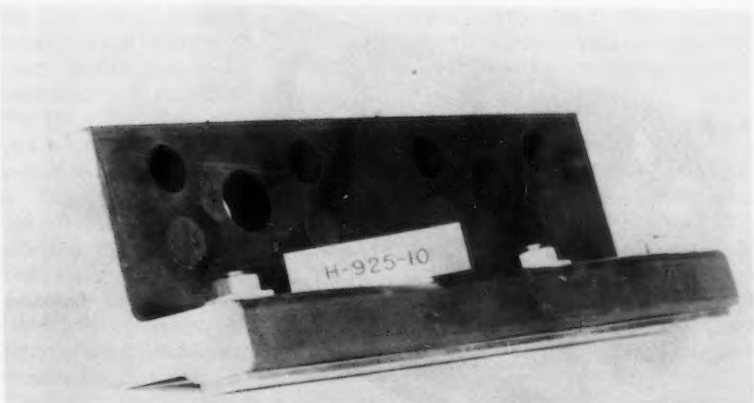
FIGURE 14

RADIAL GATE SEAL TESTS

Belt Type Side Seals with Metal Shoes



(A) View of the angle seal with fabric reinforcement.



(B) View of the all-rubber angle seal with a brass bar added to reduce frictional drag on wall plate.

FIGURE 15

RADIAL GATE SEAL TESTS

Views of the Molded Angle Side Seal with Fabric Reinforcement and a Brass Bar

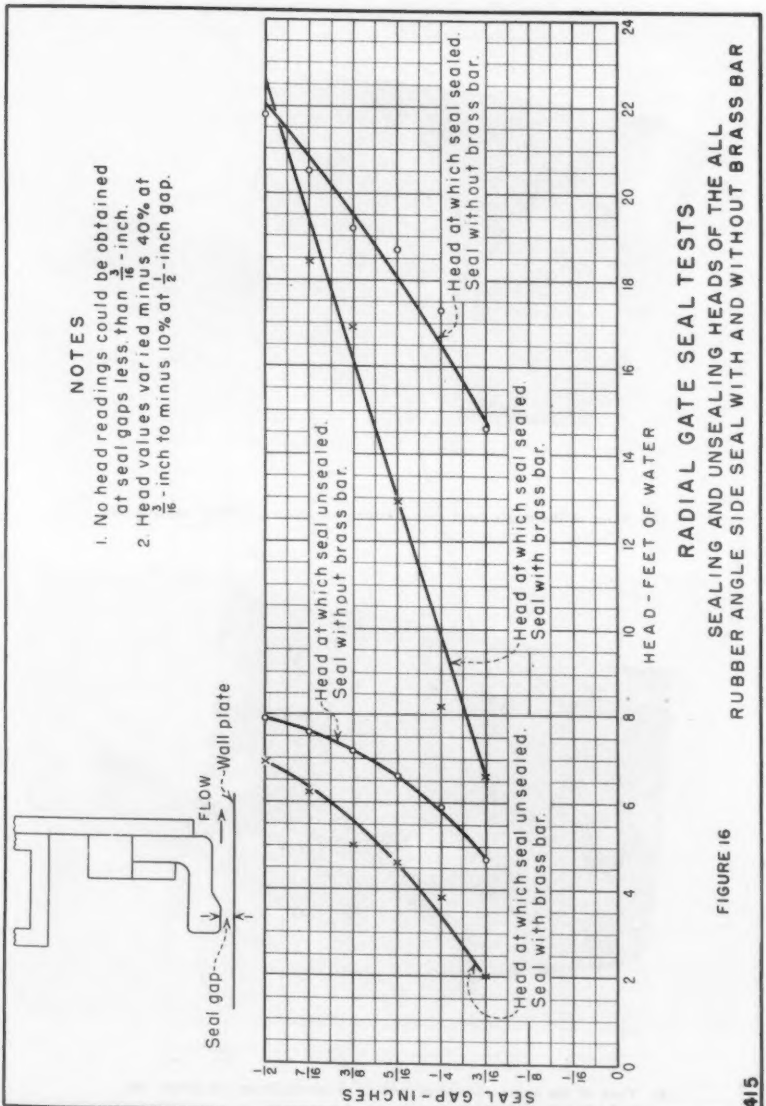
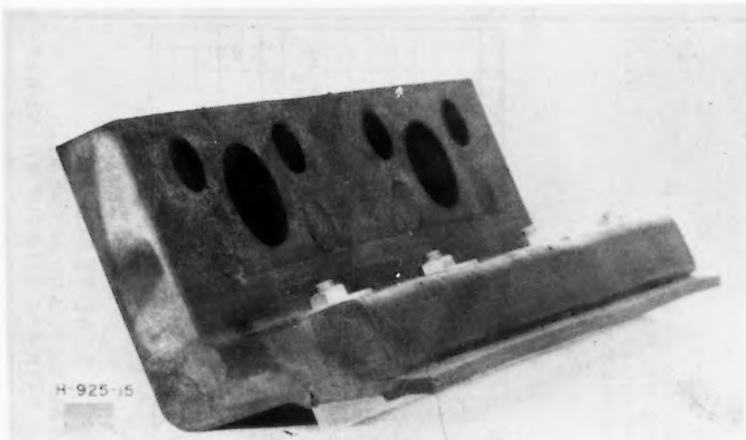


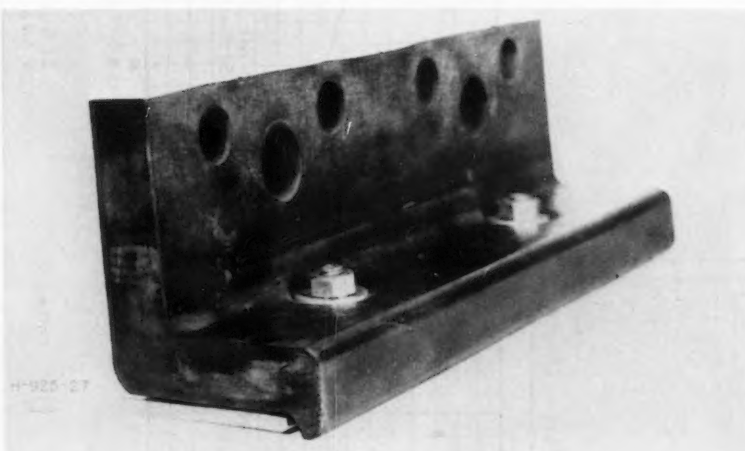
FIGURE 16

RADIAL GATE SEAL TESTS

SEALING AND UNSEALING HEADS OF THE ALL RUBBER ANGLE SIDE SEAL WITH AND WITHOUT BRASS BAR



(A) View of the angle seal with a brass bar plus a rubber sealing flap.

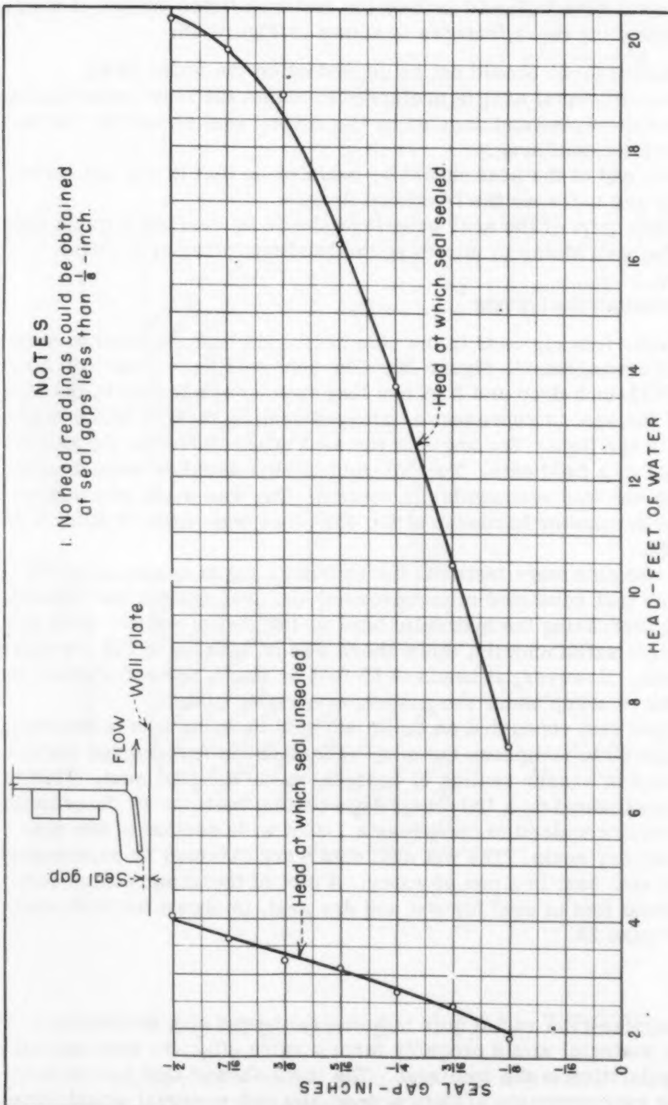


(B) View of the angle seal with rubber removed from the lower leg and a brass plate added.

FIGURE 17

RADIAL GATE SEAL TESTS

Views of the Angle Side Seal with a Brass Bar plus a Rubber Sealing Flap and with a Brass Plate.



RADIAL GATE SEAL TESTS

SEALING AND UNSEALING HEADS OF THE ALL RUBBER ANGLE SIDE SEAL WITH BRASS PLATE ADDED

FIGURE 18

415

Conclusions

Based on the results of these tests and on the computations for frictional drag, a side seal design should include the features listed below. A suggested design incorporating these features is shown in Figure 19.

1. The sealing force should not be dependent on the water head.
2. A rubber-to-metal seal is preferred to obtain the least water leakage. The portion of the waterload carried by the rubber seal should be low so as to reduce the frictional drag.
3. The heel end of the seal should be mounted so that it will not contact the wall plate and increase the frictional drag.
4. The major part of the seal waterload should be carried through a metal member to the wall plates to minimize the frictional drag of the seal.

Radial Gate Bottom Seal Tests

The hydraulic test rig used in the side seal tests was modified to simulate a bottom seal arrangement, Figure 20. The seal seat was made with a curved depression 3/32-inch deep and 3 inches long measured parallel to the long dimension of the seal to represent a damaged seating surface which might be encountered in the field. The angle of the seal seat relative to the seal was similar to that of a field gate. Two all-rubber seal samples were available for test; each one was rectangular in section. One was made of stock with Type A Shore durometer hardness of 69; the other was made of stock with a hardness of 38.

Both seal samples were tested in the hydraulic rig at heads up to 195 feet of water. This test consisted of compressing the seal against the simulated gate seat and increasing the hydraulic head to 195 feet of water. Both seals appeared to seal satisfactorily, since there was no leakage at the depression in the gate seat. However, at heads of 55 feet or more, the soft rubber seal had a tendency to creep under the downstream clamp plate.

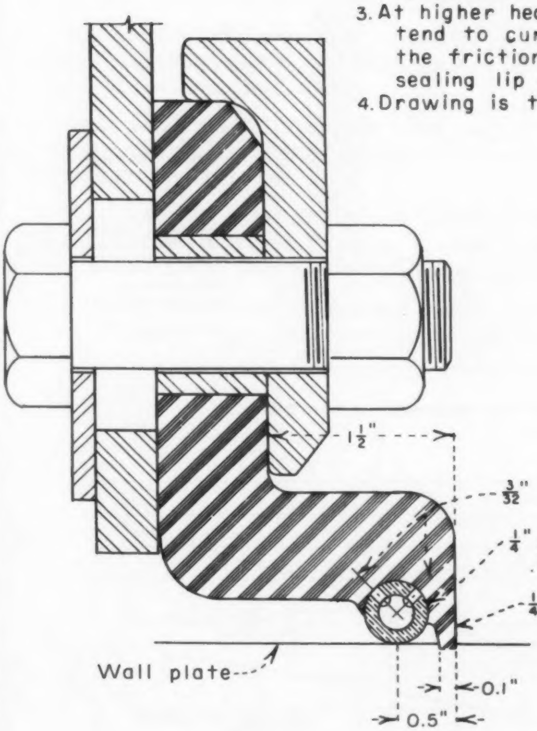
The principal test conducted on these two seal samples was to determine the force required to compress the seal. These data were desired for gate design purposes to assure sealing of depressions in the seal seat. This information was obtained on a Universal-type testing machine by determining the vertical force required to compress a 1-foot wide section of the seal against wet and dry seats. The wet seal data were obtained by submerging the simulated seal seat in a pan of water. A plot of the compression force, pounds per lineal foot of seal for wet and dry seat, is shown for both seal samples in Figure 20.

Conclusions

The rectangular-type radial gate bottom seal tested was satisfactory. The softer rubber material would probably form a more effective seal against surface irregularities in the seal seat. The tests showed that for surface irregularities approximately 3/32 inch deep, the soft material would form a seal at an applied force of 50 pounds per lineal foot of seal; the harder material would require a force of 140 pounds per lineal foot.

NOTES

1. Approximately one-third of seal water load across the $1\frac{1}{2}''$ dimension is carried by the gate; the remainder of water load is carried to the wall plate.
2. Brass pipe may be slightly curved during assembly to fit gate curvature.
3. At higher heads, seal leg will tend to curve, and thus releave the frictional drag of rubber sealing lip on wall plate.
4. Drawing is to full scale.

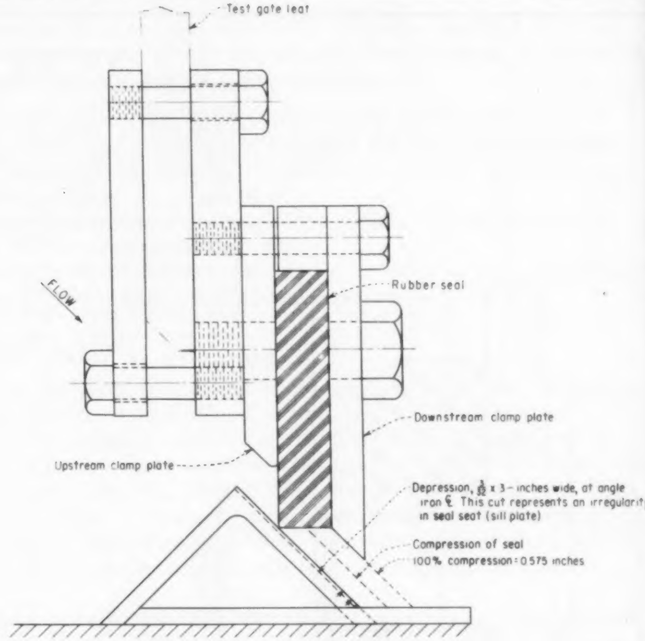


RADIAL GATE SEAL TESTS

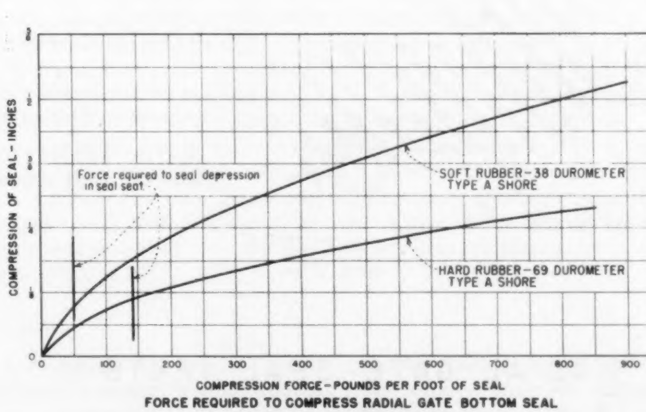
PROPOSED SIDE SEAL WITH A BRASS PIPE AND A RUBBER SEALING LIP

415

FIGURE 19



SIDE VIEW - ASSEMBLY - 12 INCHES WIDE



FORCE REQUIRED TO COMPRESS RADIAL GATE BOTTOM SEAL

RADIAL GATE SEAL TESTS

ARRANGEMENT FOR HYDRAULIC AND COMPRESSION TESTS OF BOTTOM SEAL

FIGURE 20

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BOUNDARY-SHEAR STRESS IN UNSTEADY TURBULENT PIPE FLOW

M. R. Carstens,[1] A.M. ASCE and John E. Roller,[2] J.M. ASCE

INTRODUCTION

There are two reasons for the preponderance of experimental and theoretical knowledge concerning steady flow in circular pipes as compared with unsteady flow. First, problems involving steady or quasi-steady flow in pipes arise repeatedly in the majority of engineering works. Second, experimental research of unsteady flow has been delayed until the development of reliable dynamic-pressure-measuring instruments.

In this paper, a theoretical analysis of boundary-shear stress for unsteady turbulent flow in a pipe is given, followed by a report of experiments with unsteady flow in a smooth pipe.

There is good reason to restrict the study to turbulent flow. In laminar flow, the instantaneous mean velocity and instantaneous velocity profile (hence, the boundary-shear stress) are dependent upon the history of the motivating force or piezometric-pressure gradient. As a result of this historical dependence, each case of unsteady laminar flow is unique. Conversely, in turbulent flow, the strong lateral diffusion of the turbulent eddies tends to eliminate the dependence of the velocity distribution upon the history of the motion. If this hypothesis is justified, then the dependent flow variable, boundary-shear stress, is a function of instantaneous values of the flow variables, mean velocity and acceleration (or piezometric-pressure gradient). Thus, a greater degree of generalization of results appears to be possible for unsteady turbulent flow in a pipe than for laminar flow.

Note: Discussion open until July 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1945 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. HY 2, February, 1959.

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NOMENCLATURE

A - cross-sectional area of pipe;

C_m - $\frac{1}{A}\int_A \left(\frac{v}{V}\right)^2 dA$, linear-momentum-correction coefficient;

D - inside diameter of pipe;

f - Darcy-Weisbach coefficient or boundary-drag coefficient;

g - acceleration of gravity;

h - piezometric head;

h_o - piezometric head in the reservoir;

L - length of pipe

n - reciprocal of the exponent in the power law of velocity distribution;

p^* - piezometric pressure;

Q - discharge or volume rate of flow;

r - radial coordinate measured from the centerline of the pipe;

r_o - pipe radius;

ρ - fluid density;

τ - shear stress;

τ_o - boundary-shear stress;

t - time, measured from the instant of initiation of the flow;

v - velocity in the pipe;

V - mean velocity in the pipe;

V' - "observed" velocity determined from the jet coordinates;

$\forall$ - volume of a fluid element within the pipe;

x - axial coordinate measured from the upstream end of the pipe;

u - subscript designating unsteady flow; and

s - subscript designating steady flow.

Theoretical Analysis

The linear-momentum equations are the beginning point of a rational analysis of boundary-shear stress. In uniform unsteady flow in a pipe, the velocity, v, is everywhere axial and is a function of r and t only. Similarly, the shear stress, τ, is a function only of r and t. Since there is no mean transverse acceleration, the piezometric pressure, p_u^*, is a function only of x and t.

The fluid element, of length dx, extends to the pipe wall, as shown in Figure 1. The linear-momentum equation for this element is

$$-\frac{\partial p^*}{\partial x}\, dx\, r_o^2 \pi - \tau_{ou} 2\pi r_o\, dx = \frac{\partial}{\partial x}\left[\int_A \rho v\, dQ\right] + \frac{\partial}{\partial t}\int_{\forall} \rho v\, d\forall$$

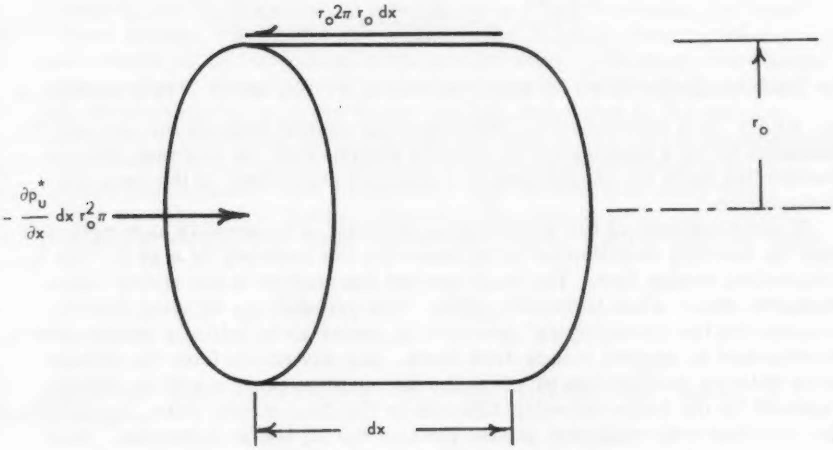

Figure 1. Fluid Element.

The terms on the left side of the equation are the external forces on the fluid element. The first term is the combined pressure and weight force in the x direction. The second term is the boundary-shear force.

The first term on the right side of the equation is the difference of momentum flux through the end areas of the fluid element at a given time. Since the equation is being applied to uniform flow, this term is zero. The existence of turbulent velocity fluctuations has no effect on the magnitude of this term.

The second term on the right side of the equation is the time rate of change of linear momentum of the particles within the interior of the element. Since dx is independent of r and since v is not a function of x, the following simplifications are possible,

$$\frac{\partial}{\partial t}\int_{\forall} \rho v \, d\forall = \frac{\partial}{\partial t}\left[\int_A \rho v \, dA\right] dx = \rho \frac{dQ}{dt}\, dx = \rho r_o^2 \pi \frac{dV}{dt}\, dx$$

Again the existence of turbulent velocity fluctuations has no effect on the magnitude of the term.

The simplified linear-momentum equation for the fluid element of Figure 1 is

$$-\frac{\partial p_u^*}{\partial x} - \frac{2\tau_{ou}}{r_o} = \rho \frac{dV}{dt} \tag{1}$$

Another linear-momentum equation can be derived in a similar manner with an annular cylindrical element of radius r and thickness dr. The resulting linear-momentum equation is

$$-\frac{\partial p^*_u}{\partial x} - \frac{1}{r}\frac{\partial (r\,\tau_u)}{\partial r} = \rho\frac{\partial v}{\partial t} \tag{2}$$

In Equation (2) $\partial p^*_u/\partial x$, is not a function of r; but, since v is a function of r, $\partial v/\partial t$ is a function of r. Therefore the second term on the left side of Equation (2) is a function of the velocity distribution. In contrast, the corresponding term for steady flow is a constant regardless of the velocity distribution.

If the magnitude of the acceleration is small, a reasonable assumption is that the velocity distribution is the same for the unsteady flow as for the corresponding steady flow. The basis for the assumption is the strong lateral diffusive effect of the turbulent eddies. The process can be visualized by considering the chronological sequence of events as an initially steady flow is accelerated to another steady-flow state. Any deviations from the steady-state velocity distribution which occur during acceleration will be rapidly removed by the turbulent-eddy diffusion in the final steady state. In addition, the turbulent-eddy diffusion is also present during the acceleration. Thus, for turbulent flows undergoing moderate acceleration, equality of the unsteady-state and steady-state velocity distributions is a rational assumption.

The power law of velocity distribution for turbulent flow in a smooth pipe is used in the subsequent analysis. The power law of velocity distribution is (1).

$$\frac{v}{V} = \frac{(2n+1)(n+1)}{2\,n^2}\left(1 - \frac{r}{r_0}\right)^{1/n}$$

The value of n is seven for pipe Reynolds numbers less than $1\,(10^5)$. The value of n increases with increasing Reynolds number, becoming 10 at a Reynolds number of $3.2(10^6)$.

Equation (2) is integrable with respect to r upon substitution of the power law of velocity distribution. The piezometric-pressure gradient is eliminated by means of Equation (1). The resulting expression for shear-stress is

$$\tau_u = \tau_{ou}\left(\frac{r}{r_0}\right) + \left[F_1\left(\frac{r}{r_0}\right)\right] r_0\,\rho\,\frac{dV}{dt} \tag{3}$$

and for shear-stress gradient is

$$\frac{\partial \tau_u}{\partial r} = \frac{\tau_{ou}}{r_0} + \left[F_2\left(\frac{r}{r_0}\right)\right]\rho\,\frac{dV}{dt} \tag{4}$$

Values of $F_1\,(r/r_o)$ and $F_2(r/r_o)$ are given in the Table for $n = 7$. The shear-stress gradient is a function of r in contrast to steady flow in a pipe.

The unsteady-flow shear-stress in Equations (3) and (4) would be more meaningful if related to the well-established expressions for shear stress in steady flow. This relation is possible by virtue of the equality of the shear stress and the shear-stress gradient at the centerline for both steady and unsteady flow. Since similar velocity distributions were assumed in steady and unsteady flow, the deviation of unsteady-flow shear-stress distribution from

linear must be solely the result of acceleration. If all fluid particles were accelerated equally, as in rigid-body motion, the internal stresses (shear stresses) would not be influenced by the acceleration. Therefore, the change in the shear-stress distribution must be the result of relative acceleration between adjacent particles in the cross section. Inasmuch as the velocity distribution is symmetrical about the pipe axis and is continuous through the axis, the relative acceleration between a particle on the axis and a particle an infinitesimal distance from the axis will also be infinitesimally small. Thus, as a consequence of the assumption of identical velocity distributions in unsteady and steady flow, the shear-stress gradients at the axis are equal, or

$$\left.\frac{d\tau_s}{dr}\right|_{r=0} = \left.\frac{\partial\tau_u}{\partial r}\right|_{r=0}$$

In addition, the shear stress is zero at the pipe axis for either steady or unsteady flow, or

$$\tau_s\Big|_{r=0} = \tau_u\Big|_{r=0} = 0$$

But in steady flow

$$\frac{d\tau_s}{dr} = \frac{\tau_{os}}{r_o}$$

Introducing these concepts into Equation (4),

$$\frac{\tau_{ou}}{r_o} = \frac{\tau_{os}}{r_o} - \left.F_2\left(\frac{r}{r_o}\right)\right|_{r=0} \rho\frac{dV}{dt} \qquad (5)$$

Similarly, for Equation (3),

$$\tau_u = \tau_{os}\left(\frac{r}{r_o}\right) + \left[F_3\left(\frac{r}{r_o}\right)\right] r_o\,\rho\frac{dV}{dt} \qquad (6)$$

Values of $F_3/r/r_o)$ are given in the Table for $n = 7$. The physical concepts embodied in Equations (4), (5), and (6) are more easily visualized by referring to Figure 2.

Equation (5) is the desired relationship between the boundary-shear stress in unsteady flow and that in steady flow. Rearranging Equation (5) and introducing the boundary-drag coefficient, $f = 8\tau_o/\rho V^2$, results in

$$\frac{f_u}{f_s} = 1 + 0.449\,\frac{D}{f_s V^2}\,\frac{dV}{dt} \qquad (7)$$

The boundary-drag coefficient, f_u, is influenced in two ways by the magnitude of the Reynolds number. First, f_s is a function only of the Reynolds number for flow in a smooth pipe. Second, the value of n, which characterizes

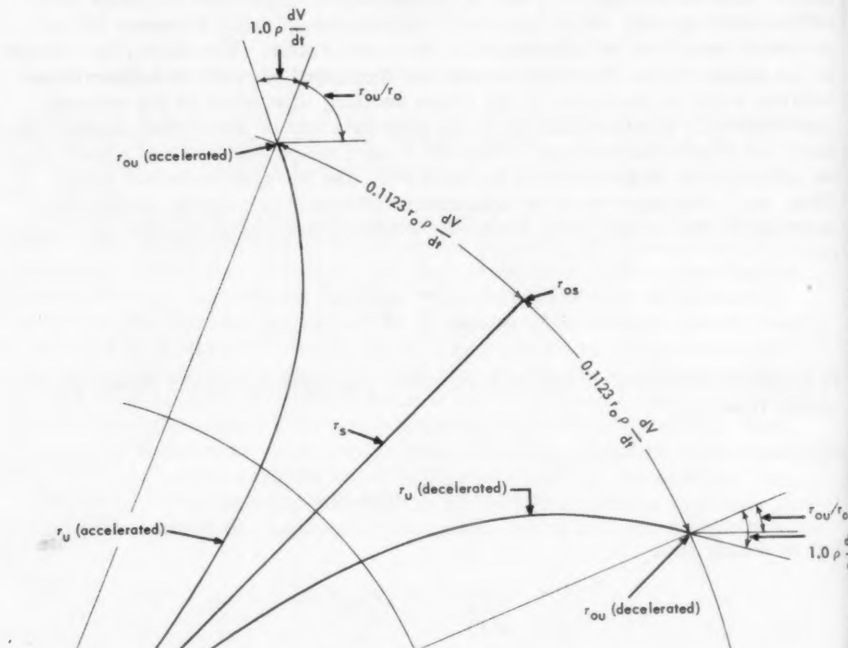

Figure 2. Shear Relationships in Unsteady Turbulent Flow.

the shape of the velocity profile, is also a function of the Reynolds number. The numerical value, 0.449, in Equation (7) is for n = 7. This numerical value is 0.391, 0.346, and 0.310 for n of eight, nine, and ten, respectively.

Thus, the theoretical analysis is indicative that the boundary-shear parameter, f_u, of unsteady and turbulent flow in a smooth pipe is independent of the history of the motion and is dependent only upon the corresponding steady-state value of f_s and the instantaneous value of the acceleration parameter, (D) (dV/dt) / V^2. Equation (7), which is a mathematical expression for this dependence, was formulated using the hypothesis that the velocity distribution is similar for both steady and unsteady flow. For moderate acceleration, the hypothesis appears to be valid, but experimental results are desirable in order to verify and to establish the limits of the hypothesis.

TABLE: Values of F_1, F_2, and F_3 for $n = 7$

r/r_o	$F_1(r/r_o)$ Equation (3)	$F_2(r/r_o)$ Equation (4)	$F_3(r/r_o)$ Equation (6)
0	0	-0.1123	0
0.1	-0.011	-0.0962	0.0002
0.2	-0.0200	-0.0861	0.0025
0.3	-0.0279	-0.0706	0.0058
0.4	-0.0340	-0.0534	0.0109
0.5	-0.0385	-0.0320	0.0176
0.6	-0.0403	-0.0080	0.0261
0.7	-0.0396	0.0256	0.0378
0.8	-0.0349	0.0706	0.0549
0.9	-0.0224	0.1414	0.0786
1.0	0	1.0000	0.1123

Experimental Study

Whether steady or unsteady flow, experimental determination of boundary-shear stress is indirect. In unsteady flow, this determination, by means of Equation (1), is complicated by the necessity of measuring $p^*(x,t)$ and $V(t)$. In these experiments, the function $V(t)$ was determined by analysis of a motion-picture record of the free jet of the pipe outlet. Commercially available pressure transducers were used to determine $p^*(x,t)$.

The boundary and initial conditions of the experiment were as follows: (a) the smooth, straight, circular pipe was attached to a large reservoir of water in which the piezometric head was maintained constant; and (b) the flow was initiated by the instantaneous opening of a downstream valve. Since the fluid was at rest prior to the opening of the valve, the initial flow was laminar throughout the pipe. With the passage of time, spots of turbulence were generated. These turbulent spots increased in size with increasing time until the flow was turbulent throughout the pipe.

Experimental Equipment and Technique

The overall arrangement of the equipment is shown in Figure 3. The test section was extruded 1/2-inch brass pipe. The experimentally determined piezometric-head gradient of six steady-flow runs was within the range, +0.1 per cent to -1.08 per cent, of the piezometric-head gradient computed from the Blasius equation. This close agreement means that the pipe was smooth and that the diameter, measured by means of a micrometer, was correct. Piezometers were spaced at intervals of 95 diameters along the pipe. The pressure-transducer signals were amplified electronically and were recorded on a Sanborn two-channel oscillograph. The configuration of the downstream jet from the horizontal pipe was recorded by means of a 35 mm motion-picture camera.

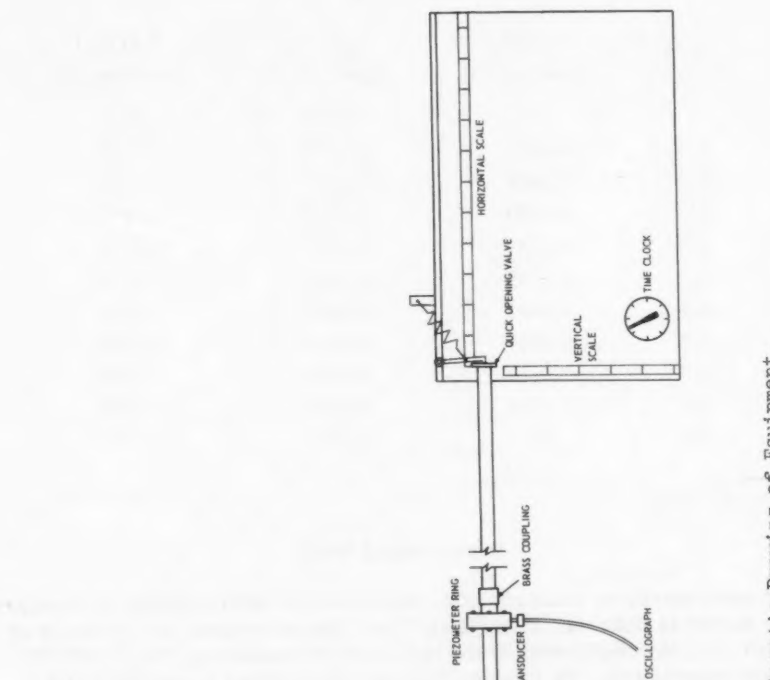

Figure 3. Schematic Drawing of Equipment.

Experimental Data

The oscillographic records of pressure were replotted as instantaneous piezometric-head lines in order to delineate the laminar- and turbulent-flow regions. The succession of piezometric-head lines of a typical run are shown in Figure 4. The abscissa of Figure 4 is the distance along the pipe axis from the reservoir. The ordinate is h/h_o. The position of the zero value of the ordinate is lowered with increasing values of $t\sqrt{gh_o}/L$ in order to avoid the superposition of many piezometric-head lines. The dashed line in Figure 4 delineates the regions of laminar and turbulent flow. Laminar-flow regions are above the dashed line and turbulent-flow regions are below the dashed line.

With the free-fall equations, the "observed" velocity-time relationship of the downstream jet was determined from the coordinates and from the time recorded on the 35 mm film strip. Two assumptions are implicit in the use of the free-fall equations. First, air resistance is negligible on the jet. Second, the jet is composed of individual masses in which each fluid particle has the same linear momentum at the point of measurement. Two experimental observations substantiated the validity of the assumptions, as follows: (a) the "observed" velocity-time relationships were identical whether the point of measurement was 0.5 ft below the pipe outlet or whether the point of measurement was 2.50 ft below the pipe outlet; and (b) in all the film strips, discrete masses of fluid appeared to be forming in the jet at the point of measurement. However, the "observed" velocity, V', is not the mean velocity in the pipe.

The mean velocity, V, in the pipe is less than the "observed" velocity, V', determined from the jet coordinates. The difference in the two velocities is due to the radial variation of velocity within the pipe. The radial variation of velocity at the pipe outlet is the result of the boundary-shear force as the fluid passes through the pipe. As the fluid passes the pipe outlet, the boundary-shear is decreased suddenly to a negligible value and the internal shear forces within the jet tend to eliminate the non-uniform velocity distribution. The linear momentum of the jet is greater at the pipe outlet than if the velocity distribution were uniform. The mass of fluid passing the pipe outlet in a time Δt is $\rho Q \Delta t$. The linear momentum of this fluid mass is $C_m \rho Q V \Delta t$. The linear-momentum-correction coefficient, C_m, is a measure of the non-uniformity of the velocity distribution as the fluid passes the pipe outlet. In the absence of any horizontal external forces the linear momentum of this mass is constant in the jet. At the point of measurement of jet coordinates, the linear momentum is $\rho Q V' \Delta t$. Therefore, the desired quantity V is equal to V'/C_m. Only the value of the "observed" velocity, V', can be determined from the film strip of the jet. Other methods had to be employed to determine C_m.

The experimentally determined values of $\partial h/\partial x$ of laminar flow were incorporated into the solution of the Navier-Stokes equations in order to determine the velocity distribution at the pipe outlet as a function of time. It is to be noted in Figure 4 that the flow was laminar at the pipe outlet for $t\sqrt{gh_o}/L < 1.38$ even though turbulent flow existed in portions of the pipe at a much earlier time. The heat diffusion equations analogous to the Navier-Stokes equations have been solved with analogous boundary and initial conditions and with the analogous piezometric-head gradient as an arbitrary function of time.(2) The piezometric-head gradients were determined from

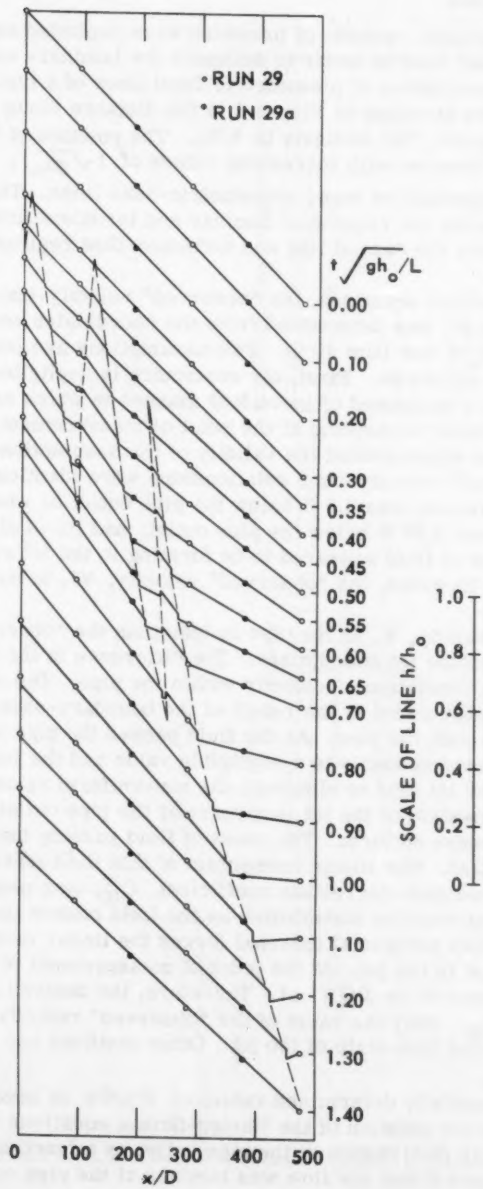

Figure 4. Piezometric-Head Lines of a Typical Run.

Figure 4 and were replotted in Figure 5. A simple analytical function, $\partial h/\partial x \sim \cos Kt$ was selected as being a good approximation for the experimentally determined piezometric-head gradient of laminar flow. Of course, K had to be determined empirically for each of the eight runs. This function was then incorporated into the solution of the Navier-Stokes equations for the determination of C_m as a function of time. Because the length of the laminar-flow region was less than the distance between pressure measurement stations when $t\sqrt{gh_o}/L > 1.00$, the piezometric-head gradient of laminar flow could not be experimentally determined from Figure 4 when $t\sqrt{gh_o}/L > 1.00$. Thus during the interval $1.00 < t\sqrt{gh_o}/L < 1.38$, C_m had to be determined in some other manner.

The magnitude of the discontinuity of V' at $t\sqrt{gh_o}/L = 1.38$ shown in Figure 6 was utilized to obtain the value of C_m at this time. The discontinuity occurs at the time the turbulent flow first passes the outlet. The difference in velocity distribution between the turbulent flow and preceding laminar flow results in a discontinuity of linear momentum in the laminar and turbulent jets at this time even though V is continuous. The value of C_m of the turbulent flow was assumed to be 1.02 based upon the seventh-root

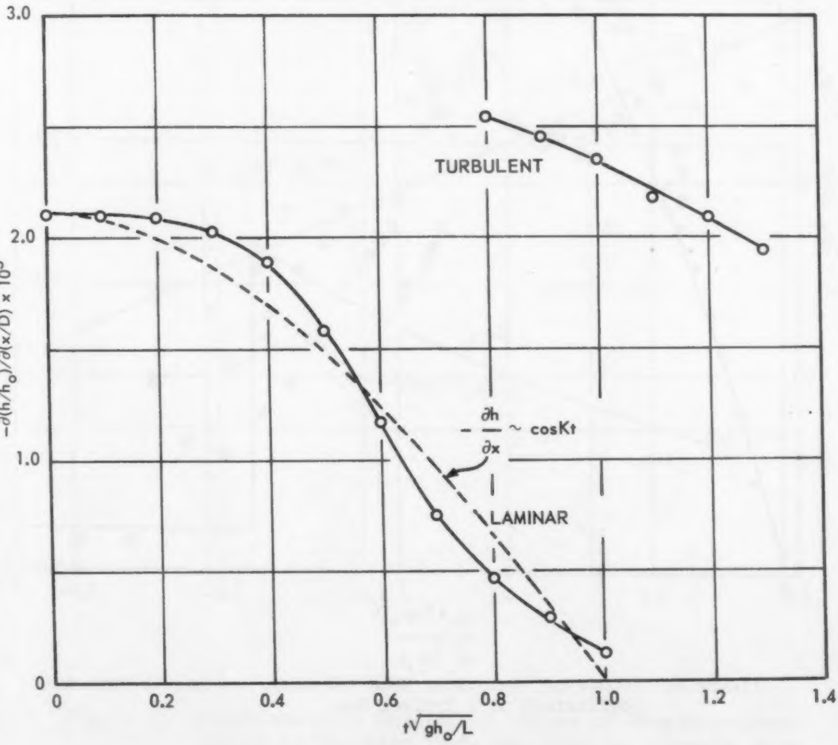

Figure 5. Piezometric-Head Gradients of a Typical Run.

law of velocity distribution. Since V' must equal $C_m V$ and since V is equal for both laminar and turbulent flow at this time, the experimental point, shown as a circle in Figure 6 was readily determined. The dashed line in Figure 6 is an interpolation between the two methods.

The resulting V-t function is shown in Figure 6.*

Experimental Results

The boundary-shear stress was determined indirectly by means of Equation (1) utilizing the data shown in Figures 5 and 6. The results are

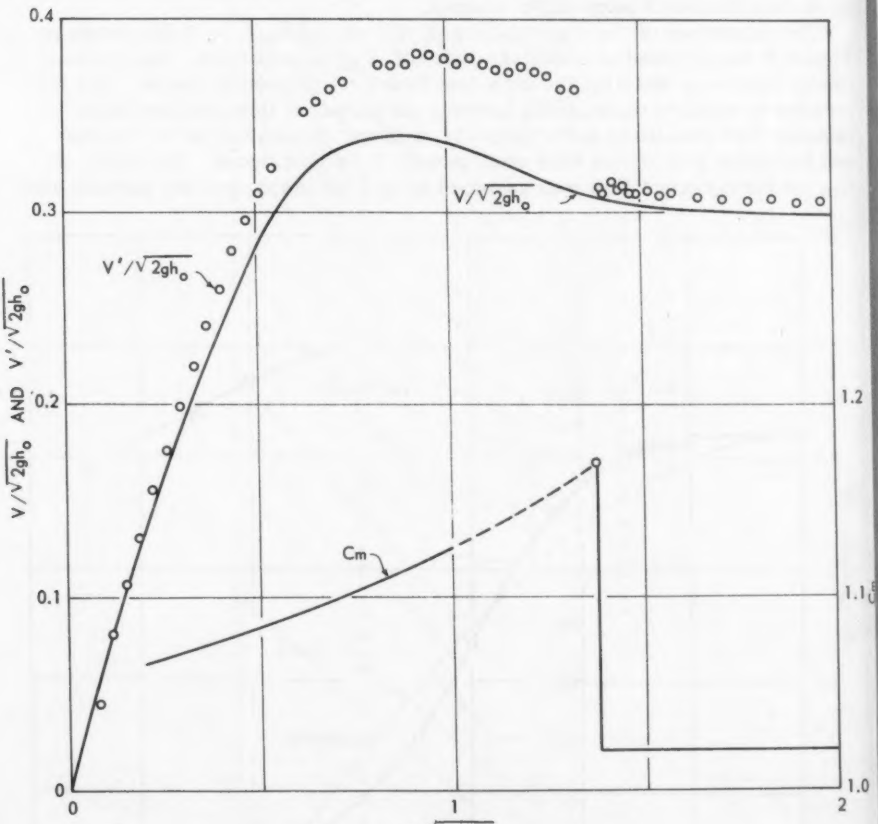

Figure 6. "Observed" Velocity, Mean Velocity, and Linear-Momentum Coefficient of a Typical Run.

*A more complete discussion of the method of analyzing the data is contained in the paper "Transition from Laminar to Turbulent Flow in a Pipe," by M.R. Carstens, Journal of the Hydraulics Division, Proc. A.S.C.E., Paper 1450, December, 1957.

presented in Figure 7 in the form suggested by Equation (7). Results of experiments conducted at Massachusetts Institute of Technology[3] in an unsteady-flow water tunnel are also shown in Figure 7.

Analysis of Results

There are two obvious explanations for the scatter of the experimental results shown in Figure 7.

First, the scatter of the experimentally determined values is an indication of the precision of determining the drag coefficient, f_u. The p-t function could be measured within a probable error of ± 2 per cent in regard to pressure. In view of the complexities in determining the V-t function from the motion-picture data, a probable error of ± 3 per cent would seem to be

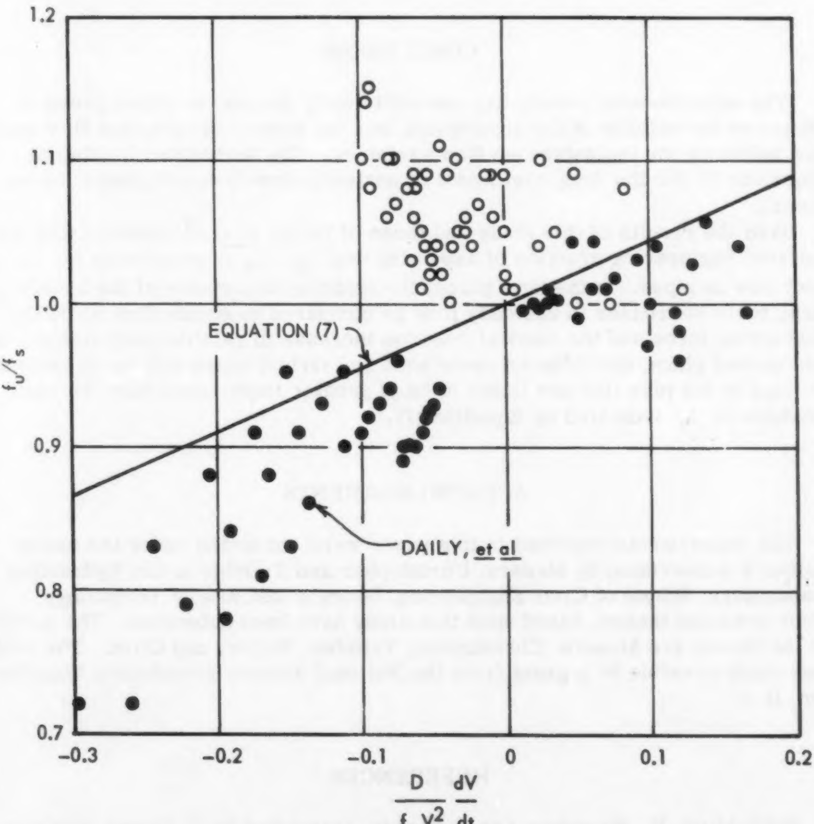

Figure 7. Experimentally Determined Values of Boundary-Shear Stress of Unsteady Turbulent Flow in a Smooth Pipe.

optimistic. A further source of possible error is in the graphical determination of dV/dt from graphs similar to Figure 6. Since none of these possible errors is compensating, the entire scatter of the experimental results might be attributed logically to experimental error.

Second, the unsteady-flow drag coefficient, f_u, may be a function of the history of the motion. The presentation of f_u/f_s as a function of $D(dV/dt)/f_sV^2$ is deduced from Equation (7). However, the basic assumption in the derivation of Equation (7) is that the history of the turbulent flow does not influence the instantaneous flow variables. Thus, if history of the unsteady motion is important, the scatter of the experimental results in Figure 7 might be the result of omitting the parameters necessary to define this history. In this regard, Daily, et al[3] noted a historical effect during deceleration but not during acceleration.

CONCLUSIONS

The experimental results are not sufficiently precise to either prove or disprove the validity of the assumption that the history of turbulent flow does not influence the instantaneous flow variables. The derivation, leading to Equation (7) for the drag coefficient of unsteady flow is considered to be rational.

Both the results of this study and those of Daily, et al,[3] indicate that the current engineering practice of assuming that $f_u = f_s$ is acceptable for turbulent flow in pipes. In the first place, the relative importance of the boundary-drag force decreases in unsteady flow as compared to steady flow while the disturbing force and the inertial reaction increase in relative importance. In the second place, the unknown resistance and virtual-mass effects of various fittings in the pipe line are likely to be of greater importance than the small changes in f_u indicated by Equation (7).

ACKNOWLEDGMENTS

The experiments reported in this paper were conducted under the senior author's supervision by Messrs. Christopher and Trimble in the Hydraulics Laboratory, School of Civil Engineering, Georgia Institute of Technology. Four graduate theses, based upon this study have been submitted. The authors of the theses are Messrs. Christopher, Trimble, Roller, and Olive. The study was made possible by a grant from the National Science Foundation, Washington, D. C.

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CHARACTERISTICS OF FLOW OVER TERMINAL WEIRS AND SILLS[a]

Corrections to Discussion by Corrado Ruggiero[1]

CORRECTIONS—In printing F. Paderi's discussion of this paper in Proceedings Paper 1832 (October, 1958) the fifteen references used in that discussion were inadvertently omitted. We offer them at this time for the benefit of interested readers:

All these branches of research offer several developing possibilities, as it is attested by the authors surveys, which are very valuable on either the experimental side or because of reflections on the theoretical field.

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THE HYDRAULIC JUMP AT AN ABRUPT DROP[a]

Closure by Walter L. Moore and Carl W. Morgan

WALTER L. MOORE,[1] A.M. ASCE and CARL W. MORGAN,[2] A.M. ASCE.—The authors wish to thank Mr. McPherson and Mr. Forster for their valuable discussions.

Mr. McPherson has compared and integrated the findings with those of others. He asks about the influence of the direction of movement of the tailwater level on the limiting depths for the various types of jumps. A brief investigation of the effect of rising and falling tailwater, made at the time of the first experiments, indicated a negligible effect of this variable provided sufficient time was allowed for equilibrium to be established. As a result of the comments by Mr. McPherson and the data he presented additional experiments on this effect were made in the Hydraulic Laboratory at the University of Texas. These experiments indicated that if sufficient time is allowed the effect of the direction of tailwater movement was of the same magnitude as the experimental error in determining the tailwater depth for Froude numbers less than 6. In fact for $\frac{\Delta Z_o}{Y_1} = 3$ and $F_1 = 4$ and also for $\frac{\Delta Z_o}{Y_1} = 4$ and $F_1 = 6$ it was possible to set the tailwater so that the jump would slowly alternate between the B type and the wave type. In order to change from the B type to the wave type or vice-versa in a moderate time it was necessary to set a higher value with rising tailwater than with falling tailwater, a result similar to those observed by others. Those results, however, were for a different range of F_1 and relative drop height $\Delta Z_o/Y_1$ than used in the authors' experiments. The equilibrium condition which the authors obtained for $F_1 < 6$ could not be obtained at $F_1 = 8$. For $\Delta Z_o/Y_1 = 4.0$ and a rising tailwater the change from the B type to the wave type required a $\frac{Y_2}{Y_1} = 12.3$ while for a falling tailwater the change occurred at $\frac{Y_2}{Y_1} = 11.4$. It is felt that this may have been due to the fact that at this Froude number the top of the wave rose higher than the walls of the flume. On the other hand it may be that at this high Froude number the flow pattern is such that the "hysteresis effect" mentioned by McPherson and others is present.

In the additional experiments mentioned above, tailwater depths were ascertained by means of piezometers rather than with a point gage as in the original experiments. The experiments with $\frac{\Delta Z_o}{Y_1} = 3$ were made at a scale

a. Proc. Paper 1449, December, 1957, by Walter L. Moore and Carl W. Morgan.

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twice that used in the original experiments. These new tests indicated that the values shown in Fig. 7 for the maximum jump B are slightly high. Fig. 7a is here presented to better define the several values required to insure a jump B or a wave to form.

Mr. McPherson calls attention to the disagreement with data presented by Hsu(4,5) in which a systematic transition between the types of jump (A, B, Wave) was found as a function of the Froude No. F_1. Actually the investigation reported by the authors was started to explore some longitudinal characteristics of the jump at a drop assuming the existence of the systematic transition reported by Hsu. It was soon found that type B jumps could be obtained at low values of F_1 where Hsu's curves indicated only jump A would form. Further investigation showed that the full range of jump types was possible at any value of F_1 (at least above 2.0) controlled only by the tailwater elevation. The systematic transition as a function of Froude No. was not observed by the authors.

From considerations similar to those leading to figure 13 it is apparent that a tailwater control which produced a relatively flat curve of tailwater elevation vs discharge as opposed to the steep one shown in Fig. 13 would result in a systematic transition from Jump B through the wave to Jump A as the discharge was decreased. With the depth Y_1 held constant a reduction in discharge would correspond to a reduction in F_1. This could explain a systematic change of jump type with decreasing F_1 as reported by Hsu. However, it is not clear how the slope of the curve of tailwater depth $\frac{Y_2}{Y_1}$ vs Froude No. F_1 could be positive for the type A & B jumps and negative for the wave type jump as indicated in his Fig. 10 of Reference 5.

Mc. McPherson's Fig. D-2 calls attention again to the fact that the band shown for Jump A in Figs. 6, 7, and 8 does not represent the maximum values of the tailwater. In the Results section of the paper it is stated that "The conditions under which Jump A will form are shown as a very narrow band, but actually Jump A will form at values of Y_2/Y_1 greater than the upper bound------."

In response to Mr. McPherson's suggestion values of Y_2/Y_1 are listed below for the various forms of the jump for which relative bottom velocities are presented in Figs. 9, 10, 11.

Mr. Forster effectively emphasizes the importance of the type of jump (type A, wave, or type B) in determining the severity of attack on the channel bottom just downstream from the drop. His examples well illustrate the effectiveness of Jump A and the wave in protecting the bottom from high velocity currents. However, the wave type can result in some fairly high bottom velocities for large drops ($\frac{\Delta Z_o}{Y_1} = 3$, $\frac{\Delta Z_o}{Y_1} = 4$ Figs. 10 and 11). In Figures 9, 10, and 11 the various regions and curves are identified by a letter and a number as listed in the table. It can be seen that with the wave type and a Froude number of 4 a downstream velocity of $3V_2$ may be attained for $\frac{\Delta Z_o}{Y_1} = 3.0$ and $4V_2$ for $\frac{\Delta Z_o}{Y_1} = 4.0$. It may seem odd that a Froude number of 4 resulted in higher relative bottom velocities than Froude numbers of 2, 6, and 8. This does not seem unreasonable, however, when the complex flow pattern below the drop is considered. With a large $\frac{\Delta Z_o}{Y_1}$ and a small Froude

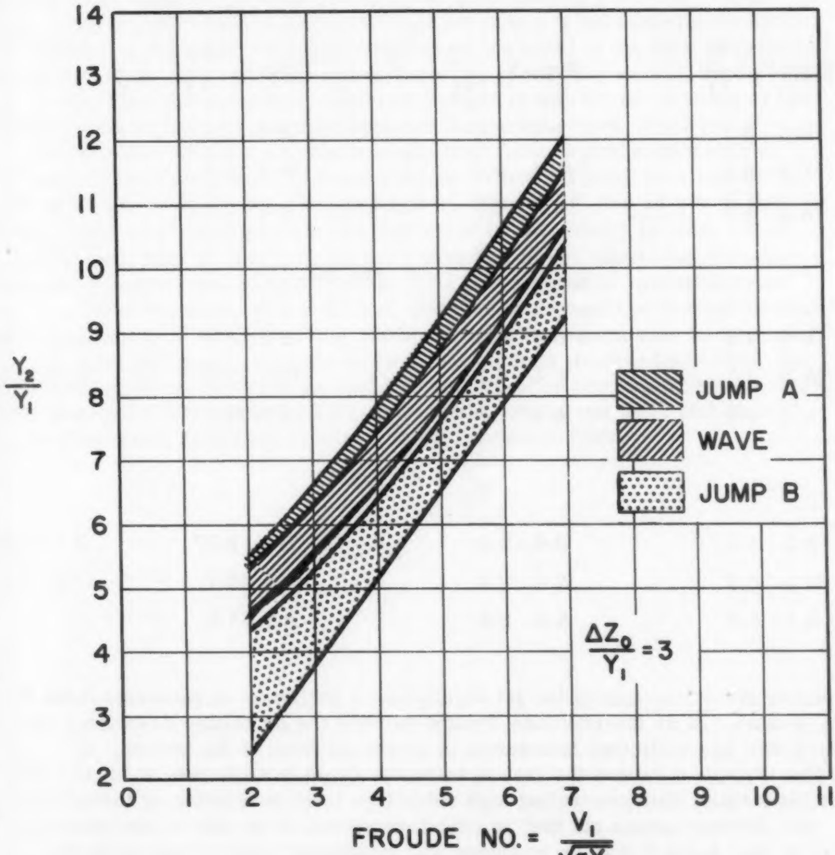

FIG. 7a FORMS OF THE HYDRAULIC JUMP AS A FUNCTION OF FROUDE NO. AND RELATIVE DOWNSTREAM DEPTH

$\frac{Z_o}{Y_1} = 2$ Fig. 9

Type F_1	$\frac{Y_2}{Y_1}$	Type F_1	$\frac{Y_2}{Y_1}$	Type F_1	$\frac{Y_2}{Y_1}$
B 2	3.1	B 4	5.4	B 6	8.1
W 2	4.3	W 4	6.8	W 6	9.4
A 2	4.6	A 4	7.1	A 6	9.9

$\frac{Z_o}{Y_1} = 3$ Fig. 10

B 2	4.1	B 4	6.3	B 6	8.4	B 8	11.4
W 2	5.1	W 4	7.2	W 6	10.4	W 8	12.4
A 2	5.4	A 4	7.8	A 6	10.6	A 8	13.5

$\frac{Z_o}{Y_1} = 4$ Fig. 11

B 2	4.2	B 4	5.9	B 6	8.80	B 8	10.5
W 2	6.3	W 4	8.5	W 6	10.7	W 8	11.8
A 2	6.5	A 4	8.9	A 6	11.4		

number the momentum of the jet simply is not sufficient to penetrate down to the bottom. At an intermediate Froude number the jet falling down from the wave still has sufficient momentum to penetrate down to the bottom. At higher Froude numbers the rise in tailwater depth is sufficient to lift the jet off the bottom, thus preventing high velocities from developing on the bottom.

Mr. Forster points out that an overdimensioned drop may be unconservative in that Jump B may be produced with associated high bottom velocities. This is certainly true for the conditions he assumed; that is, that Y_2/Y_1 is constant. This is equivalent to assuming that the size of the drop is controlled by the elevation of the upstream channel while the downstream channel elevation and tailwater depth remain fixed. If, on the other hand, the size of the drop were controlled by lowering the bottom elevation of the downstream channel for a suitable distance while the tailwater elevation remains constant as controlled by conditions farther downstream, an oversize drop will then be conservative. Using the same initial conditions as in Mr. Forster's example ($F_1 = 2.8$ and $Y_2/Y_1 = 5$) and a drop of $\Delta Z_o = 2Y_1$, Fig. 6 will give a minimum wave type jump (bordering Jump B). If the drop is increased to $\Delta Z_o = 3Y_1$, by lowering a suitable length of the downstream channel bottom, the values become $F_1 = 2.8$, $Y_2/Y_1 = 6$, yielding from Fig. 7a a maximum wave type jump (bordering on Jump A). In this manner the low bottom velocities would preserved even with an oversize drop.

Although Figs. 9, 10, and 11 give some very valuable data indicating the range of velocities to be expected near the channel bottom, they are not intended to be used for precise estimates of velocities in particular instances. Due to the complex nature of the flow pattern there is necessarily considerable scatter in the data. In addition the scour potential of the flow is certainly dependent upon the turbulence characteristics of the flow as well as the mean velocity. Thus the velocities should be thought of merely as an index of the scour potential for comparisons between the various types of jumps.

Mr. Forster points out in his example that Jump B produces a bottom velocity of the same order of magnitude as a drowned jump on a flat apron. The peak values of the bottom velocities in Figs. 9, 10, and 11 are of the order of $6V_2$ which is the same as that given by Mr. Henry (Trans. ASCE, 1950, p. 690, Fig. 33) for a moderately drowned jump. These relative high velocities are not necessarily harmful if a paved apron is provided downstream from the drop for a sufficient distance to protect the channel bottom. For a situation in which a rising discharge and consequent rise in tailwater would cause increasing submergence of the jump as its size increases, the introduction of an abrupt drop would be beneficial by preventing severe drowning. Instead of drowning the jump would shift toward the wave and Jump A with consequent reduction of attack on the channel bottom.

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FLOOD FREQUENCIES DERIVED FROM RAINFALL DATA[a]

Closure by J. L. H. Paulhus and J. F. Miller

J. L. H. PAULHUS,[1] M. ASCE, and J. F. MILLER[2].—In the November (1958) Journal of the Hydraulics Division, in Proceedings Paper 1856, there appears a discussion by C. O. Clark of Proceedings Paper 1451 that the authors were unaware of when they prepared their closing remarks to the discussion by Messrs. Riggs and Benson (Proceedings Paper 1690, June, 1958). The following remarks, therefore, are the closing remarks to Mr. Clark's discussion and form an addendum to the closure published in November:

Generally speaking, the differences between Clark's opinions and those of the authors are of a philosophical as well as statistical nature. The writer criticizes the authors' use of the term "unusual" in describing the 1933 Codorus Creek flood. Clark claims that the most intense 6-hour rainfall over 10 square miles in the storm producing this flood, i.e., 6.8 inches, is not a sufficient approximation to the estimated probable maximum value of 24 inches to be called unusual. Is that a measure of unusualness? Methuselah is reputed to have lived 900 years. Hence, following Clark's reasoning, it would not be unusual for someone to live 200 years!

The results of a recent extensive rainfall-frequency study(1) involving all available precipitation records in the Middle Atlantic region indicate that the shortest average recurrence interval that could reasonably be assigned the 6.0 inch, 10-square mile, 6-hour rainfall of the October 1942 storm for either Codorus or Passage Creek basins would be about 160 years. Now, there is no reason why a rainfall of that intensity should always occur when the surface layers of the soil are, practically speaking, saturated, as they were in the case of the October 1942 Passage Creek flood. The same rainfall intensity could occur under much less favorable flood-producing soil conditions. Consequently, the shortest average recurrence interval of the October 1942 flood can reasonably be expected to be longer than 160 years, indicating that the writer's flood-frequency curve for Passage Creek probably yields excessive peak discharges for the longer recurrence intervals. Examination of the streamflow records reveals that the curve very likely yields excessive discharges for the shorter recurrence intervals also. For example, the curve shows a two-year peak discharge of about 6000 cfs although that value has been equaled or exceeded only three times in the 26-year period, 1932-57 (see Basin D in Table 1 below).

a. Proc. Paper 1451, December, 1957, by J. L. H. Paulhus and J. F. Miller.
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The shortest average recurrence interval that might reasonably be assigned the 7.4 inch, 10-square mile, 6-hour rainfall of the 1899 storm mentioned by the writer is about 800 years. The peak discharge of 36,000 cfs that Clark estimates could happen on Codorus Creek from such a rainfall under optimum flood-producing conditions would very likely have a longer average recurrence interval than 800 years. It should be realized, however, that even if the average return period of such a flood were 1000 years, 1 percent of the intervals between occurrences might be less than 10 years, and 5 percent might be less than 51 years.(2) Obviously, the occurrence on Codorus Creek of a peak discharge of this magnitude within the next few years should not be astonishing. Nevertheless, the event may still be considered unusual.

Now, how about the unusualness of the August 1933 maximum peak discharge of 11,200 cfs observed on Codorus Creek? This was still the maximum observed for the 25-year period ending with water year 1957. According to the laws of probability there is a 1-percent chance that the average return period of this flood peak could be over 2000 years, and a 5-percent chance that it could be about 500 years.(2) Consequently, the 11,200 cfs peak discharge could be an unusual event.

The chief trouble with Clark's line of reasoning is that there is no objective way for determining when or where to stop. For example, the Smethport, Penn., storm of July 17-18, 1942 (OR 9-23) produced the maximum 10-square mile, 6-hour rainfall ever observed in the United States, namely, 24.7 inches. For some reason, the writer does not mention this storm. Perhaps it was believed that rainfall of that intensity could not occur over either Codorus Creek or Passage Creek basins. Why not? The probable maximum rainfall for both basins is estimated to be somewhat higher than for the storm site. Presumably then, rainfall of equal or greater intensity than for Smethport storm could occur or could have occurred over these two basins. The fact remains, however, that there is no evidence of the occurrence of such a rainfall on either of these basins within the period of our recorded precipitation or streamflow history, and the authors know of no way to base frequency analysis on what "could happen."

Referring to Passage Creek, Clark states, "This stream had its floods; Codorus Creek did not. At least not in the period 1933-54. In the 26-year period 1932-57, Passage Creek experienced three peak discharges exceeding the maximum for Codorus Creek. Is it not possible that Codorus Creek is less prone to flooding than Passage Creek? On the other hand, is it not possible that Passage Creek, through chance distribution, has received more than its share of floods for this 26-year period? Table 1 suggests that the latter is definitely possible. Passage Creek is the only basin for which the three highest annual flood peaks are 3 to 5 times the fourth highest. A frequency analysis (Gumbel method adjusted for partial duration series) of its annual floods for the 26-year period, 1932-57 yields the following:

Return period (years)	2	5	10	25	50	100
Peak discharge (1000 cfs)	4.5	8.5	11.5	15.5	18.6	21.5

These data differ from Clark's frequency curve, presumably based on regional considerations, by less than 10 percent except for return periods shorter than about 5 years. Comparison of the statistical arrays of annual floods for the seven basins of Table 1 indicates that the Passage Creek record

Table 1. STATISTICAL ARRAY OF ANNUAL PEAK DISCHARGES (100 CFS)

FOR SEVEN BASINS OF 61 to 101 SQUARE MILES

Rank	Basin						
	A	B	C	D	E	F	G
1	97	112	41	210	150	160	97
2	94	61	41	147	73	137	92
3	50	49	36	123	65	115	79
4	43	42	34	41	36	112	77
5	42	32	34	39	29	110	73
6	42	32	30	38	28	110	69
7	41	31	30	37	27	95	66
8	38	27	30	32	26	70	59
9	35	27	29	31	26	70	57
10	33	25	28	30	24	70	47
11	29	24	27	26	23	68	39
12	27	22	25	26	23	64	35
13	26	22	24	23	23	64	32
14	24	19	22	21	22	62	32
15	24	19	21	20	22	62	26
16	24	18	20	18	21	60	18
17	22	18	19	18	20	49	17
18	21	17	18	17	20	39	17
19	21	16	17	16	20	38	8
20	19	16	16	14	17	36	
21	17	14	16	14	17	32	
22	16	14	16	13	15	27	
23	15	11	11	12	14		
24	14	11		12	12		
25	14	9		6	12		
26	8			5	12		
27					10		

A. Chester Creek near Chester, Penn., 61 sq. mi., 1932-57
B. Codorus Creek at Spring Grove, Penn., 74 sq. mi., 1933-57
C. Linganore Creek near Frederick, Md., 82 sq. mi., 1935-57
D. Passage Creek at Buckton, Va., 87 sq. mi., 1932-57
E. Seneca Creek at Dawsonville, Md., 101 sq. mi., 1931-57
F. Tohickon Creek near Pipersville, Penn., 97 sq. mi., 1936-57
G. Tye River near Lovingston, Va., 92 sq. mi., 1939-57

provides about the most unrepresentative sampling for the group. Can Clark's curve then be accepted as representative for this group of basins?

The authors were unable to determine the plotting formula used or the criteria for construction of the curve in the writer's frequency plot. Neither could it be determined why the frequency curve ascribed to the authors was plotted from their Table 4. For the Codorus Creek flood peaks plotted, the curve should have been drawn from data in the authors' Table 3. Neither set of data is recommended as representative for Codorus Creek. For the situation under discussion, the authors recommend the use of all available streamflow records for the basin supplemented as much as possible by synthesized flood peaks. However, the synthesized flood peaks for the full period of rainfall record should yield good results. Table 2 compares these results with those from the observed record and shows that the difference is appreciable. The data were taken from Tables 3 and 6 of the authors' original paper.

Table 2. FLOOD FREQUENCIES (100 CFS) – CODORUS CREEK

(By Gumbel Analysis – Not Adjusted for Partial Duration Series)

	Return Period (years)					
	2	5	10	25	50	100
Observed record, 1933-54	25	48	64	83	97	112
Synthetic record, 1900-54	22	38	49	62	72	82

In view of Clark's questioning the reliability of flood frequencies derived from something other than floods and from data outside the watershed, the authors were astonished by his reference to rainfall data outside the subject basins as an indication of the possible frequency of extreme values of observed peak discharges. Furthermore, the manner in which Clark uses streamflow and/or rainfall data within a large region to establish flood frequencies for a small basin suggests a tendency to confuse the frequency of occurrence of an extreme event within the region with the probability of its occurrence on the small basin.

In conclusion, next to streamflow measurements, rainfall data are logically the best index to flood frequencies. The idea of regional procedures is sound, but there is no reason why such procedures based on the relatively short records of streamflow generally available should be reliable for the longer recurrence intervals. The authors' procedure could be used to increase synthetically the length of streamflow records for individual basins to provide a more reliable base for regional procedures that would yield much more dependable results, especially for the longer recurrence intervals, than is now possible.

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TURBULENCE CHARACTERISTICS OF THE HYDRAULIC JUMP[a]

Closure by Hunter Rouse, T. T. Siao,
and S. Nagaratnam

HUNTER ROUSE,[1] M. ASCE, T. T. SIAO,[2] and S. NAGARATNAM[3].—Comments upon this paper were of particular interest in that they reflected the views of investigators at five of the principal hydraulics laboratories now engaged in research on turbulent flow. Significant points already considered during the preparation of the paper were emphasized, and others not considered or at least not given their proper attention were very pertinently noted. As is always the case, the net result is a marked improvement over the original contribution.

The writers recommend that everyone who has access to a glass-walled flume carry out the experiment described by Mr. Silberman - artifically varying the velocity distribution, if necessary, by the application of a coarse bed roughness. The formation of a standing eddy on a level floor is as convincing a demonstration of a basic boundary-layer principle as one could hope to find. Other hydraulic distortions of the jump proper have been discussed in detail elsewhere, but they might well be mentioned here in this connection: those due to variation of channel cross section, particularly through the trapezoidal range; to variation of bed slope, as in a chute, bed curvature, as on a spillway bucket, or bed alignment, as at the end of an apron; and variation in wall alignment or curvature, as in flared or warped chute spillways. Each of these will have an influence upon the jump characteristics just as important in its way as that of bed roughness - but just as inessential to the basic jump phenomenon. In conducting the investigation, the writers attempted to eliminate every possible extraneous factor - including initial velocity distribution - so that full attention could be given to the mechanism of turbulent expansion. Surely the introduction of a sonic throat and shock wave, as suggested by Mr. Silverman, would not have been an effective step in this direction.

Messrs. Peterka and Bradley introduced a welcome note into the literature in their immediate recognition of the usefulness of academic findings in clarifying the practical aspects of a related problem. The question of jump length has long been a matter of controversy, in part for lack of agreement on nomenclature and in part because the jump actually never ends but only

a. Proc. Paper 1528, February, 1957, by Hunter Rouse, T. T. Siao and S. Nagaratnam.

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approaches asymptotically a state of depth, velocity, and turbulence equilibrium. Designation of an end is thus purely arbitrary. Messrs. Peterka and Bradley have realistically adopted a criterion based upon bed velocity; Bakhmeteff and Matzke logically based theirs on depth; the writers conveniently terminated their boundary curves where the effect of further curvature seemed to be inappreciable. Were residual turbulence (the academic choice) to determine the nominal end, its position would obviously depend upon the percentage arbitrarily chosen for the permissible turbulence remainder. Since this remainder continues to be appreciable in comparison with that of the subsequent uniform flow for some distance downstream, such a definition would produce by far the greatest nominal length. The discussors are right in their belief that length of roller and length of rapid expansion would approach equality as the Froude number became very large; if the residual turbulence were related to the overall loss, all definitions would then agree at the (unfortunately, purely academic) limit.

Mr. Robertson, to select a minor comment from his discussion at this point, speaks of a normal or "proper" hydraulic jump as one which occurs in established open-channel flow. This recalls, of course, the discussion by Mr. Silberman. Messrs. Peterka and Bradley, on the other hand, may well think of a proper jump as one controlled by the baffle piers of a foreshortened apron. "Established" flow is actually no more definite than is the design of a pier and apron, for it can vary through wide limits with channel roughness and shape. For this reason Mr. Harleman's detailed momentum analysis of the jump, and his application of the resulting equation to the three conditions described in the paper, represent supplementary material of basic import. The writers fear, however, that readers will ignore the equation and assume the plotted curve to indicate a general solution. The fact should hence be emphasized that even in established flow there will be a different deviation from the elementary equation for every Reynolds number, relative roughness, and channel shape.

To return to Mr. Robertson's discussion, the writers can only report that there was no indication of bottom separation in either flume or duct for the condition of minimal boundary-layer growth that was sought experimentally. So far as the upper boundary effect was concerned, in the vicinity of the nonuniformity it was surely not one of shear; whereas a more refined method of measurement would undoubtedly have revealed a positive shear above the roller, as correctly expected by Mr. Robertson, comparison of its probably magnitude with that in the region of high velocity gradient below would not lead one to anticipate much change from the conditions shown. The effect of the boundary upon large-scale fluctuations, on the other hand, was admittedly considerable, and some indication of its magnitude is undoubtedly to be found in Mr. Hubbard's measurements.

Free-turbulence literature now distinguishes among three different regions of the turbulence spectrum: the fine-scale region, containing the eddies in which most of the energy of the turbulence is concentrated; and the large-scale region of intermittency described by Mr. Robertson. In this sense the large eddies which are so obvious a part of the true hydraulic jump represent neither an appreciable concentration of energy nor an important source of dissipation. They can, however, play a considerable role in the process of mixing. At the juncture between the roller and the main stream the intermittent formation of these large eddies (often distinguishable in jump photographs) is much like that observed at the edges of jets, wakes, and boundary

layers. The free surface, which is the more noticeable zone of intermittent fluctuation and that to which Mr. Robertson refers, has some of the same characteristics. However, as noted in the discussion, it also has two interrelated features which distinguish it: abrupt discontinuity of the liquid medium itself, and the stabilizing or restoring action of gravity. The air model surely duplicated the essential freedom of motion in the lower zone of intermittency; to what extent the fluctuations in this zone are actually controlled by the free-surface fluctuations remains to be determined.

Mr. Robertson correctly implied that the plotted dissipation functions were of a partially assumed rather than a completely measured nature, in view of the representation of the nine velocity derivatives by a single one through the relationships of isotropy. Whether this assumption requires only a high Reynolds number to be closely valid, or the components even then would deviate to different degrees in different directions and different zones of flow, must also remain to future investigations for a conclusive answer. Surely the present measurements do not indicate the existence of much variation in validity from section to section, else the production-convection-dissipation sum would have departed more radically than it did from a zero value. It should also be remarked in this regard that such indications of small-scale isotropy as exist to date have been found under conditions of free turbulence, whereas deviations therefrom have all been noted in the vicinity of solid boundaries.

Measurements of the type proposed implicitly by Mr. Robertson and explicitly by Mr. Hubbard must eventually be made, although the writers shudder to think of the painstaking labor involved in determining at many successive sections - under conditions in which even the depth itself is very hard to define - what to date has been measured only for representative sections of flows which are either uniform or similar from section to section. Mr. Hubbard apparently has at hand equipment that will accomplish some part of the measurements required - at least at a safe distance below the freely fluctuating surface - and he has demonstrated a knack of filling other needs as they arise. Still to be found are a group of expert laboratory technicians with considerable time at their disposal, and an unusually patient and sympathetic sponsor!

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SNOWMELT RUNOFF[a]

Discussion by K. R. Wright

KENNETH R. WRIGHT,[1] A.M. ASCE.—Snowmelt forecasting is usually considered as being limited to the mountainous regions of the country. In this paper the authors have shown that it can also be utilized in non-mountainous areas.

The method presented by the authors appears to be a sound approach to the problem of snowmelt forecasting, and it does present a procedure for developing similar relationships for other non-mountainous areas.

The writer has checked the reliability of the relationship

$$Y = 27.4X - 1.88X^2$$

Where X = Melt Potential

Y = % of Runoff at 7" M.P.

by applying it to three years of records in much the same manner as a river-level forecaster would use it. The three years chosen were 1953, 1954, and 1955, which were subsequent to the authors' period of study.

The quarter-day dry bulb temperature, dew point temperature, wind speed, and sky cover for the Wausau, Wisconsin, station were used to calculate the quarter-day melt potentials. The melt potentials were a summation of the convection, condensation, and radiation melt potentials, as was stated by the authors.

Precipitation was also determined on a quarter-day basis throughout each of the three spring seasons. The Medford and Marshfield, Wisconsin, stations were used to determine the precipitation. These are the two stations used by the authors. The rainfall was added directly to the total quarter-day melt potentials.

Daily discharges of the Big Eau Pleine River as given by the U. S. Geological Survey were corrected according to the "ice index theory," given in the paper, for the period of ice effect. The beginning of the melt period and the total inches of runoff were determined from these daily discharges.

The meteorlogical data was then used as a river-level forecaster would use the data to determine future flows. The forecasted flows were then checked against the actual daily river discharges. Due to the unusually small snow accumulation in 1954, however, it was necessary to resort to the authors' Table VI to forecast the snowmelt hydrograph for 1954.

a. Proc. Paper 1834, November, 1958, by J. H. Zoller and A. T. Lenz.

1. Cons. Engr., Boulder, Colo.

The forecasted hydrographs are compared to the recorded hydrographs in Figures 1 and 2 for 1953, 1954, and 1955. In 1955 the forecasted and recorded hydrographs are quite close for the first five days of the snowmelt, and then they diverge. The divergence is due to the fact that the snow on the ground had completely melted. On March 21 and 22, though, there was a new snowfall which required ripening, and which then ran off on March 30 and 31 as indicated by the hydrograph. The forecasted hydrographs were based on the runoff reaching the gaging station the day after the melt potential occurred. This is in accordance with the authors' statement that there is a 13-hour lag in runoff for the basin and also that most of the melt potential occurs in the second period of each day.

The forecasted and recorded runoff for the three years was put into mass curves and also compared. These are shown in Figures 3 and 4. The horizontal portions of the forecasted mass curves are due to there being no melt potential for those specific days. This is illustrated for 1955 from March 16 through March 19.

In Figure 9 of the authors' paper the relationship developed is presented graphically and compared with several years of reliable data. Figures 5 and 6 of this discussion give the relationship as determined for 1953 and 1955. It is shown that the equation which was developed fits well with the recorded runoff for these two years. 1954 was not compared because the very small amount of snow on the ground caused a distortion of the relationship.

The summary of the melt conditions for the three years studied for this discussion is as follows:

Year	Melt Period	Length of Melt Period	Rain During Melt Period	Inches of Runoff During Melt Period
1953	March 20 to March 28	9 days	0.12 inches	4.17 inches
1954	March 24 to March 25	2 days	0.58 inches	0.76 inches
1955	March 12 to April 1	21 days	1.15 inches	3.77 inches

The Climatological Data bulletins report that in 1953 there were 15 to 20 inches of snow on the ground before melting began. In 1954 there was only about one inch of snow, resulting in only a very small amount of runoff. The 1955 bulletin lists 3 to 4 inches of snow before melting, and then on March 21 and 22 a snowfall raised it to 8 inches.

From the writer's investigation of the three years subsequent to the authors' period of study, the following conclusions were drawn in regard to the Zoller-Lenz method of forecasting snowmelt runoff.

1. The forecast of the daily discharges for 1953, 1954, and 1955 correlated satisfactorily with the recorded discharges on the Big Eau Pleine River.

2. The relationship developed does not hold true for the runoff from unusually small snow accumulations such as in 1954.

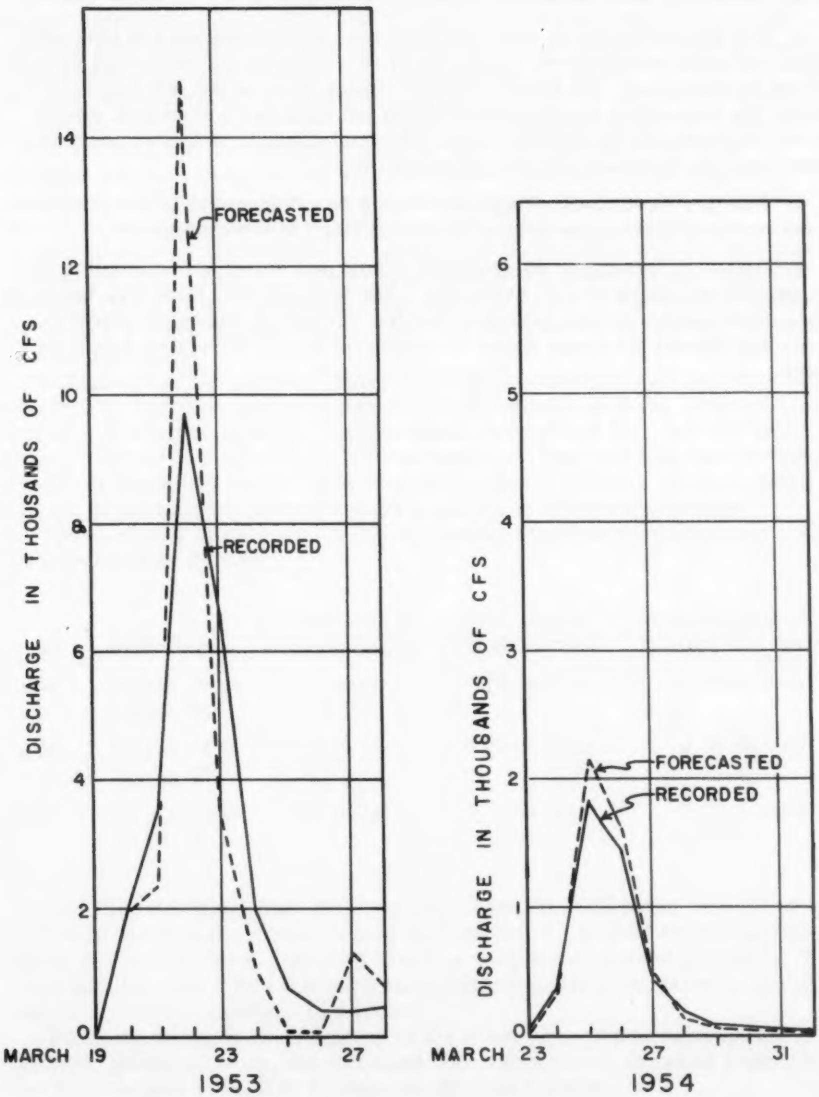
3. With a small drainage basin the size of the Big Eau Pleine Basin, the runoff forecasts can only be made for one day in advance. The U. S. Weather

Bureau's five-day forecasts cannot alleviate this to any great extent because of the amount of detailed meteorlogical data needed for the authors' method.

4. The determination of when the snow will attain ripeness and thus when additional melt potential will cause runoff is a problem which has not been solved by this paper. On small drainage basins such as the Big Eau Pleine Basin, the forecaster would probably miss the first day's snowmelt runoff. He would, however, be alerted to the ripeness by this first day's runoff and then begin his forecasts for the subsequent days.

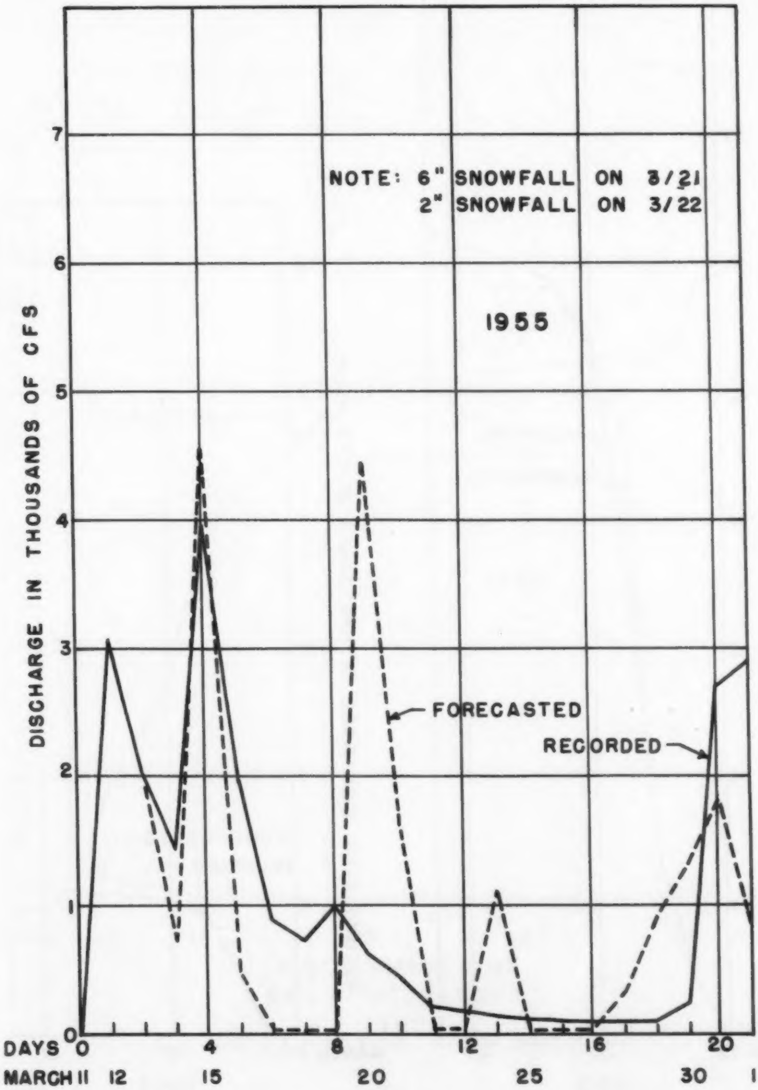
5. The "ice index theory" appears to be a sound approach to the problem of determining discharges during the rising stage of the hydrograph.

In summary, this paper on snowmelt runoff is a valuable contribution to the published information on hydrology. The fact that this study was based on a non-mountainous region indicates that the interest in snowmelt runoff is spreading beyond the areas where it comprises nearly the entire supply of water.



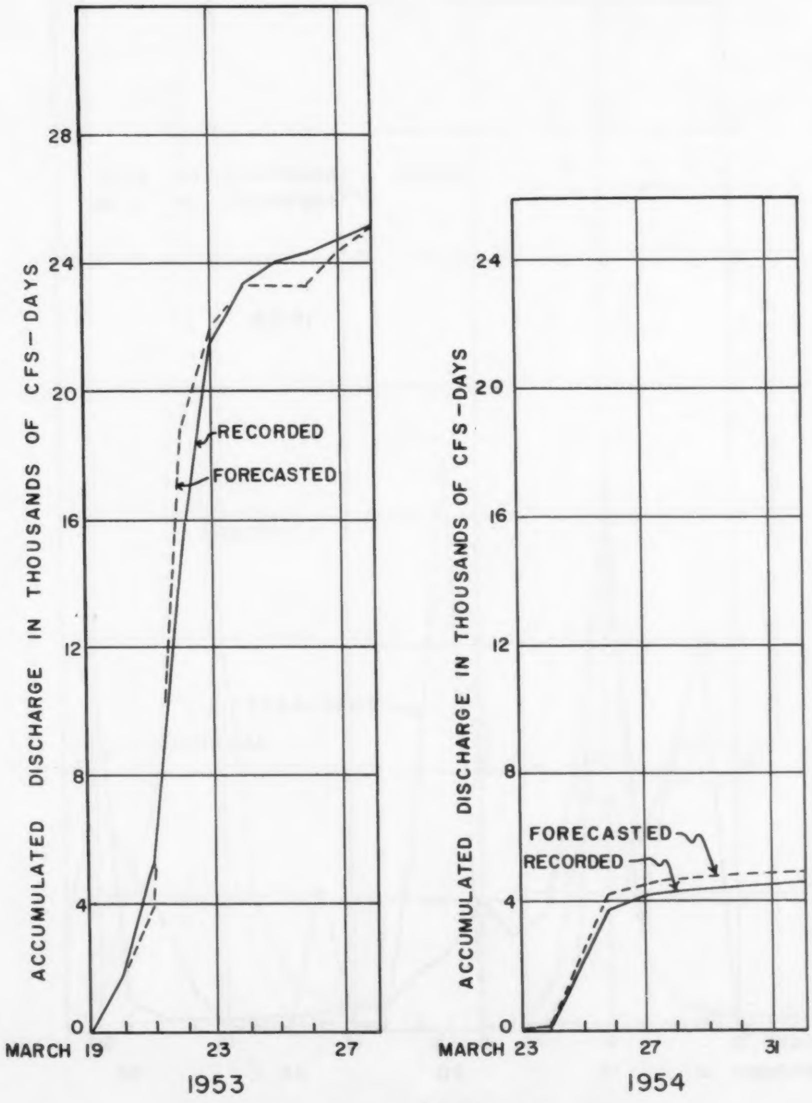
SNOWMELT HYDROGRAPHS
BIG EAU PLEINE RIVER BASIN

FIGURE 1

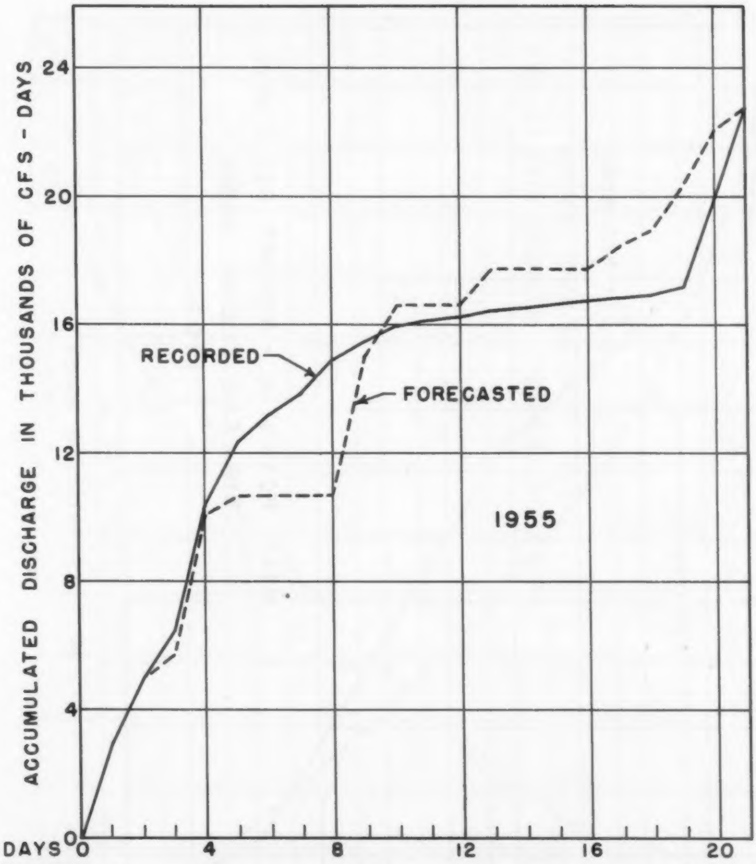


1955
SNOWMELT HYDROGRAPH
BIG EAU PLEINE RIVER BASIN

FIGURE 2



MASS CURVES OF DAILY DISCHARGES
BIG EAU PLEINE RIVER BASIN
FIGURE 3



MASS CURVES OF
DAILY DISCHARGES
1955
BIG EAU PLEINE RIVER BASIN

FIGURE 4

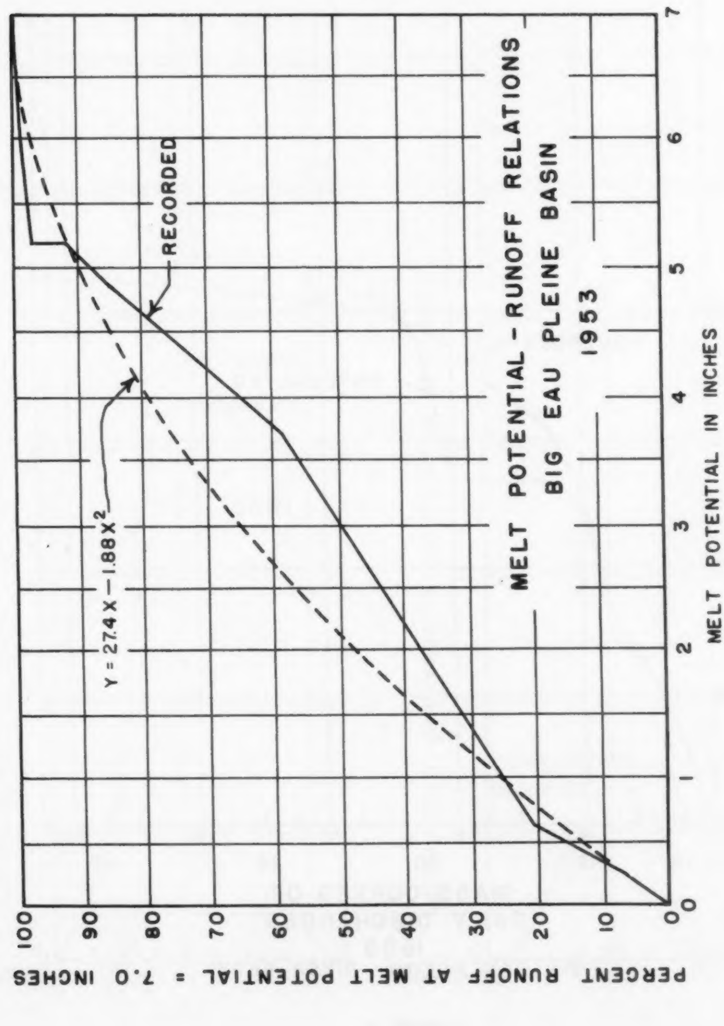

MELT POTENTIAL - RUNOFF RELATIONS
BIG EAU PLEINE BASIN
1953
RECORDED
Y = 27.4X - 1.88X^2
MELT POTENTIAL IN INCHES
PERCENT RUNOFF AT MELT POTENTIAL = 7.0 INCHES
0 1 2 3 4 5 6 7
0 10 20 30 40 50 60 70 80 90 100

FIGURE 5

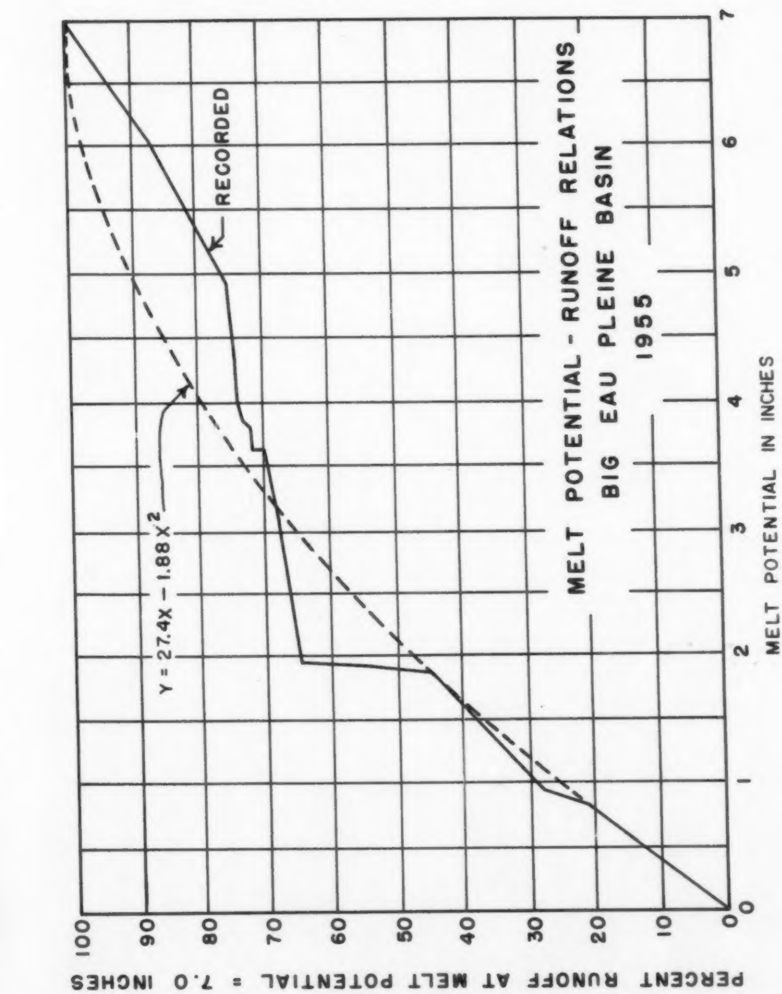

MELT POTENTIAL - RUNOFF RELATIONS
BIG EAU PLEINE BASIN
1955
RECORDED
Y = 27.4X − 1.88X²
MELT POTENTIAL IN INCHES
0 1 2 3 4 5 6 7
PERCENT RUNOFF AT MELT POTENTIAL = 7.0 INCHES
0 10 20 30 40 50 60 70 80 90 100

FIGURE 6

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

VOLUME 84 (1958)

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568(WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(PL1)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588(HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(IR2), 1594(IR2), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616(HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629(ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647(SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(PL2), 1666(PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675(PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684(HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(EM3), 1698(SU2), 1699(SU2), 1700(ST4), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710(ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719(EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733(PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742(PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775(IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784(IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806(HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815(ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(SM4), 1823(HY5), 1824(SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(EM4)[c], 1831(SM4)[c], 1832(HY5)[c].

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850(SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858(SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(EM5), 1864(EM5), 1865(ST8), 1866(ST8), 1867(ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876(WW5), 1877(CP2), 1878(CP2), 1879(ST8), 1880(ST8)[c], 1881(EM5)[c], 1882(HY7)[c], 1883(ST8)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

VOLUME 85 (1959)

JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(ST1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(HY1)[c], 1923(SA1), 1924(SA1), 1925(SA1), 1926(PL1), 1927(HW1), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958(CO1), 1959(CO1).

c. Discussion of several papers, grouped by divisions.

HY2

FEBRUARY 1959 — 9
VOLUME 85

NO. HY 2
PART 2

Your attention is invited

NEWS OF THE HYDRAULICS DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

JOURNAL OF THE HYDRAULICS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

February, 1959

PURPOSE OF THE HYDRAULICS DIVISION
(Quoted from the Official Register)

"The advancement and dissemination of knowledge relating to the occurrence of water in nature and its behavior in structures, water courses, and underground.

"In particular the field of the Hydraulics Division shall embrace meteorology and hydrology as they affect the engineer, fluid mechanics in engineering usage, and applied hydraulics as a branch of engineering science which furnishes the basis for hydraulic design and for the practical use of water in the different specialized branches of hydraulic engineering."

MEMBERS, ASCE HYDRAULICS DIVISION EXECUTIVE COMMITTEE, 1958-1959

	Term Expires
Professor Carl E. Kindsvater, Chairman George Institute of Technology Atlanta, Georgia	1960
Dr. Arthur T. Ippen, Vice-Chairman Professor of Hydraulics Massachusetts Institute of Technology Cambridge, Massachusetts	1961
Mr. Harold M. Martin Bureau of Reclamation Building 56 Denver Federal Center Denver 2, Colorado	1959

Note: No. 1959-9 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, HY 2, February, 1959.

Mr. Maurice L. Dickinson
Bechtel Corporation
4620 Seville Avenue
Vernon, California

Mr. Joseph B. Tiffany, Secretary
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631
Vicksburg, Mississippi

Mr. Earl F. O'Brien, Contact Member from ASCE 1958-59
Board of Direction
O'Brien and Gere
400 East Genesee Street
Syracuse 2, New York

COMMITTEE CHAIRMEN, ASCE HYDRAULICS DIVISION, 1958-59

Committee on Hydrology:	Mr. Charles C. McDonald Department of the Interior Geological Survey Washington 25, D. C.
Committee on Hydraulic Structures:	Mr. Glenn E. Hands P. O. Box 7088 Kansas City, Missouri
Committee on Flood Control:	Mr. Joseph I. Perrey 1330 W. Michigan Street Indianapolis, Indiana
Committee on Sedimentation:	Mr. Albert P. Gildea 751 S. Figuroa Street Los Angeles, California
Committee on Hydromechanics:	Professor John F. Ripken University of Minnesota St. Anthony Falls Hydraulic Laboratory Minneapolis 14, Minnesota
Committee on Tidal Hydraulics:	Mr. Lindsay P. Disney U. S. Coast and Geodetic Survey Washington 25, D. C.
Committee on Publications:	Mr. James W. Ball Bureau of Reclamation Building 53 Denver Federal Center Denver 2, Colorado
Committee on Research:	Professor James M. Robertson 125 Talbot Laboratory University of Illinois Urbana, Illinois

Committee on Standards:	Mr. Rolland W. Carter U. S. Geological Survey Department of the Interior Washington 25, D. C.
Committee on J. C. Stevens Award:	Professor David W. Appel University of Kansas Lawrence, Kansas
Committee on Hilgard Prize:	Mr. Albert S. Fry Tennessee Valley Authority Knoxville, Tennessee

MEETING OF THE EXECUTIVE COMMITTEE, HYDRAULICS DIVISION

A two-day meeting of the Executive Committee was held in St. Louis on November 3-4. Those attending were:

November 3 and 4

Professor Carl E. Kindsvater, Chairman
Dr. Arthur T. Ippen, Vice-Chairman
Mr. Harold M. Martin
Mr. Maurice L. Dickinson
Mr. Joseph B. Tiffany, Secretary

November 3

Mr. Glenn E. Hands, Chairman,
Committee on Hydraulic Structures

The following items have been extracted from the minutes of the meeting:

HYDRAULICS DIVISION BUDGET. Approval has been receiver from ASCE Headquarters for the following 1958-59 budget:

Technical Committee Activities	$11,200
Administrative Acitivities	2,300
TOTAL	$13,500

NEWSLETTER. Consideration was given to a headquarters request that the Newsletter schedule conform with a more frequent mailing schedule of the Journal; however, the concensus was that the Newsletter should still be published once every two months.

COMMITTEE ON THE KARL EMIL HILGARD HYDRAULIC PRIZE. A change in the wording of the purpose of this committee, submitted by Mr. Tiffany, was approved. The revised statement will appear in the 1959 Official Register.

THEME FOR BOSTON MEETING. The Committee concurred with the suggestion by Mr. Vincent, Structural Division, ASCE, regarding the adoption of the theme, "Basic Research and Structural Engineering," for our portion of the technical program at the Boston meeting.

FUNDS FOR BASIC RESEARCH. The Committee unanimously approved the resolution of the Structural Division, ASCE, proposed for presentation at the Technical Procedure Conference of 1959, in support of the establishment by ASCE of funds for basic research in engineering sciences.

NEXT MEETINGS OF THE EXECUTIVE COMMITTEE. Two additional meetings of the Executive Committee are planned for the remainder of the Society year. One meeting will be held early in 1959 to plan the Hydraulics Division organizational changes required to conform with recommendations of the Water Resources Task Committee (see below, also October 1958 Newsletter). The other meeting is planned for July at the time of the Ft. Collins conference (see below), and will result in next year's committee appointments.

ASCE - CLEVELAND CONVENTION
May 4-8, 1959

TENTATIVE HYDRAULICS DIVISION SESSIONS

(1) Sponsored by the Flood Control Committee

Presiding: Harold M. Martin, Member, Executive Committee, and Joseph I. Perrey, Chairman, Flood Control Committee, Hydraulics Division

CONSERVANCY DISTRICTS AS FLOOD CONTROL AGENCIES, C. C. Chambers, consulting engineer, Dayton, Ohio.

OPERATION OF THE MIAMI CONSERVANCY DISTRICT FOR FLOOD CONTROL, Max L. Mitchell, chief engineer, Miami Conservancy District, Dayton, Ohio.

FLOOD CONTROL AND OTHER BENEFITS OF MUSKINGUM CONSERVANCY DISTRICT, Tom C. Shuler, chief engineer, Muskingum Watershed Conservancy District, New Philadelphia, Ohio.

(2) Sponsored by the Hydraulic Structures Committee

Presiding: Harold M. Martin, Member, Executive Committee, and Glenn E. Hands, Chairman, Hydraulic Structures Committee, Hydraulics Division.

USE OF SUBMERGIBLE GATES AT NAVIGATION STRUCTURES, E. E. Abbott, and A. J. Moors, Ohio River Division, Corps of Engineers, U. S. Army, Cincinnati, Ohio.

MODEL TESTS OF SUBMERGIBLE TAINTER GATES, Thomas E. Murphy, Chief, Hydraulic Structures Section, U. S. Waterways Experiment Station, Corps of Engineers, U. S. Army, Vicksburg, Mississippi.

RELATIVE PERFORMANCE OF CURRENT METERS IN GAGING THE DISCHARGE OF THE OUTFALL RIVERS OF THE GREAT LAKES, F. Wayne Townsend, and Frank A. Blust, Hydraulics and Hydrology Branch U. S. Lake Survey, Corps of Engineers, U. S. Army, Detroit, Michigan.

(3) Sponsored by the Hydromechanics Committee

Presiding: Harold M. Martin, Member, Executive Committee, and John F. Ripken, Chairman, Hydromechanics Committee, Hydraulics Division

FLOW CONDITIONS AT THE ENTRANCE OF A PIPE, Harold R. Henry, Assistant Professor of Civil Engineering, Michigan State University, East Lansing, Michigan.

BEHAVIOR OF BUBBLES IN A VIBRATING LIQUID, Richard Skalak, of Civil Engineering, Columbia University, New York, New York.

LABORATORY DEVELOPMENT OF THE CONTRA COSTA CULVERT OUTFALL ENERGY DISSIPATOR, S. Russell Keim, Instructor, Department of Theoretical and Applied Mechanics, University of Illinois, Urbana, Illinois.

(4) Sponsored by the Sedimentation Committee

Presiding: Harold M. Martin, Member, Executive Committee, and Fred H. Larson, Past-Chairman, Sedimentation Committee, Hydraulics Division.

THE FUNCTIONS AND OBJECTIVES OF THE SEDIMENTATION SUBCOMMITTEE OF THE INTER-AGENCY COMMITTEE ON WATER RESOURCES, Warren T. Murphy, Director, Division of Flood Prevention and River Basin Programs, U. S. Forest Service, Washington, D. C.

OBSERVATIONS OF FLOOD FLOW EFFECTS ON CHANNEL BOUNDARIES, Donald A. Parsons, Hydraulic Engineer, Agricultural Research Service, U. S. Department of Agriculture, East Aurora, New York.

THE NEW ARS SEDIMENTATION RESEARCH LABORATORY, OXFORD, MISSISSIPPI—PROBLEMS AND OBJECTIVES, Russell Woodburn, Project Supervisor, Agricultural Research Service, U. S. Department of Agriculture, University, Mississippi.

HYDRAULICS DIVISION CONFERENCE - FORT COLLINS

The eighth annual conference of the Hydraulics Division will be held at Fort Collins, Colorado, July 1-3. Co-hosts are the Colorado Section and Colorado State University. Six half-day technical sessions will include papers on various hydraulic subjects, and a tour through the Hydraulics Laboratory of Colorado State University. Preceding the conference a visit to the Bureau of Reclamation Hydraulics Laboratory in Denver is planned for June 29 followed by a guided tour of the Colorado-Big Thompson on June 30. The Colorado-Big Thompson project, built by the Bureau of Reclamation, is one of the most extensive irrigation and power projects in Colorado.

Housing arrangements for individuals and families in new, modern University dormitories will be available at reasonable rates. Social events are planned and an interesting schedule is being arranged for the ladies and children. Numerous recreational attractions, including Rocky Mountain National Park, are within an hour's drive of Fort Collins. Plan to come and enjoy a Colorado vacation trip. If you wish to reserve a mountain cabin for a family vacation, write to Housing Committee, Hydraulics Conference, Civil Engineering Department, Colorado State University, Fort Collins, Colorado. Reservations for cabins should be made as early as possible because of summer tourist demand.

More detailed information is planned for the April issue of the Newsletter.

OCTOBER SESSIONS OF ASCE BOARD OF DIRECTION

During the October sessions of the ASCE Board of Direction a number of actions were taken of particular interest to members of the Hydraulics Division, as follows:

TASK COMMITTEE ON WATER RESOURCES, ASCE. The Board approved the recommendations made in the report of the Task Committee and acted to set up the coordinating Committee on Water Resources as recommended in the report (see October 1958 Newsletter).

COMMITTEE ON TECHNICAL PROCEDURE, ASCE. Appointments to the Committee for 1959 as confirmed by the Board of Direction included Professor Carl E. Kindsvater, Chairman, of the Hydraulics Division Executive Committee. The Board also acted to approve a meeting in 1959 of the Committee on Technical Procedure, in Chicago or other central location, with attendance of the coordinating committee on Water Resources.

TECHNICAL DIVISION CONTACT MEMBERS. Appointments of contact members for 1958-59 were confirmed by the Board of Direction on 16 October. Contact member for the Hydraulics Division is Mr. Earl F. O'Brien, Director representing District 3.

COMMITTEE ON SOCIETY FELLOWSHIPS, SCHOLARSHIPS, GRANTS AND REQUESTS, ASCE. This committee was created by action of the Board of Direction, 13-14 October, in an amendment to the Bylaws (Art. VII, Sect. 3(f). Members of this new committee are:

Professor John K. Vennard, Chairman	Oct. 1959
Dr. Lorenz G. Straub	Oct. 1960
Dr. Martin A. Mason	Oct. 1961
Dr. George N. Hickox	Oct. 1962
Mr. Samuel B. Morris	Oct. 1963

As a matter of interest, four of these men are members of our division, and two are past chairmen of the Executive Committee.

FELLOWSHIPS AND GRANTS

Applications for the FREEMAN FELLOWSHIP are now being accepted for the academic year 1959-1960, and can be submitted any time before March 1, 1959. Younger members of ASCE or ASME are eligible for this award.

A new fellowship grant in the field CIVIL ENGINEERING RESEARCH has just been announced by ASCE. The grant in the amount of $5,000 is made annually and is for the purpose of aiding in the creation of new knowledge for the benefit and advancement of the science and profession of civil engineering. Applicants must be members of ASCE, be U. S. citizens, and have been graduated from an accredited curriculum.

Correspondence concerning the above fellowships should be addressed to the Executive Secretary, ASCE, 33 West 39th Street, New York 18, New York.

EIGHT CONGRESS - I. A. H. R.

The Eighth Congress of the International Association for Hydraulic Research will be held at the Windsor Hotel in Montreal, Canada, during the week

beginning August 24, 1959.

All members and those interested in the work of the Association are invited to attend and to contribute to the meeting, both by presentation of papers and participation in the discussion.

The following four subjects have been selected for study in the technical sessions by the General Assembly at the Lisbon Convention in 1957.

A. Hydraulics of Gates and Valves (including vibration, forces, cavitation, ice).

B. Fundamental Hydraulics of Ship Locks.

C. Density Currents (including effects on reservoir sedimentation, hydro-electric intakes, fishing industry, etc.)

D. Air Entrainment and Air Vents.

Two seminars for more informal study will be conducted concurrently with the technical sessions on the following subjects:

1. Ice Problems in Hydraulic Structures.

2. Transportation of Material in Water. (It is intended that this seminar deal primarily with sediments but transport of other material may be included.)

In addition to the technical sessions, excursions are being planned to the St. Lawrence Seaway structures, to some of the large hydro-electric projects of the region and to hydraulic laboratories that have contributed to the development of both. In this connection it is to be noted that the St. Lawrence Seaway Project will be officially opened in 1959 and will thereafter provide passage for deep draft ocean-going vessels to the heart of the North American Continent.

A program is being planned for the ladies that will enable them to become better acquainted with Montreal, the second largest French speaking city in the world.

The official languages of the Congress will be French and English.

TECHNICAL PAPERS. Those intending to submit papers on any of the four official subjects should so advise the Canadian Committee by January 15, 1959, giving the title and a brief description of the contents of the paper in each case.

The full texts of the papers must be in the hands of the Canadian Committee not later than March 1, 1959, in order to guarantee publication and distribution in advance of the Congress.

Each paper is to include summaries not exceeding 300 words in both French and English.

Papers that have been translated into one of the official languages should be accompanied by a copy in the original language.

Every author attending the Congress will be invited to present an outline of his paper in a resume not exceeding 500 words in length.

Notice of intention to participate in either of the seminars should be received by the Canadian Committee not later than March 1, 1959. Copies of prepared contributions to either seminar should be received not later than June 1, 1959, to ensure being included in the Agenda.

PROCEEDINGS. It is expected the proceedings of the Congress will be published. These proceedings will include:

1. Papers on the four official subjects.
2. Discussion on the four official subjects.
3. Minutes of the Seminar.
4. Free papers accepted by the Executive Committee of the Council.

FEES. The fee for the Congress is $20.00 for participants and $10.00 (Canadian Currency) for the ladies, exclusive of costs of excursions. In the case of non-members, the fee will be $25.00 for participants, which include membership, for those who are eligible, of one year in the Association. (The fees for ladies remaining the same.) The range of prices per day for rooms in the Windsor Hotel is as follows:

Single room with bath	$ 8.00 to $13.00
Double room with bath	$11.00 to $17.00

The cost of meals is extra and may be expected to vary from $1.00 to $3.00, varying with the tastes of the individual.

Travel and hotel reservations may be made through Thos. Cook and Sons.

COMMUNICATIONS. Additional circulars will be issued in due course giving details of the program and instructions concerning the preparation of papers, registration, and other matters. All communications regarding attendance, participation, submission of papers, etc., should be addressed to: Mr. Leo Roy, c/o Quebec Hydro-Electric Commission, 107 Graig Street West, Montreal 1, P.Q., Canada.

MONTHLY SCIENTIFIC PUBLICATION OF INTEREST TO HYDRAULICS DIVISION MEMBERS

It will be of interest to our Division members to know that the American Geophysical Union is combining the scientific and technological material in its bimonthly TRANSACTIONS with the quarterly JOURNAL OF GEOPHYSICAL RESEARCH and will issue the JOURNAL OF GEOPHYSICAL RESEARCH as a monthly starting in 1959. This will contain much material bearing on Hydrology of interest to our members.

The subscription rate for the new JOURNAL is $16.00 for the calendar year. Memberships, however, are invited. Membership is $10.00 per calendar year and includes a subscription to the JOURNAL. Any who are interested should write to the office of the American Geophysical Union, 1515 Massachusetts Avenue, N. W., Washington 5, D. C.

FOR YOUR CALENDAR
ASCE Meetings

May 4-8, 1959	ASCE, Cleveland Convention
July 1-3, 1959	Hydraulics Division, Fort Collins Conference
October 19-23, 1959	ASCE, Washington, D. C., Convention
March 7-11, 1960	ASCE, New Orleans Convention
June 19-23, 1960	ASCE, Reno Convention

October 9-13, 1960	ASCE, Boston Convention
April 10-15, 1961	ASCE, Phoenix Convention
October 16-20, 1961	ASCE, New York Convention
February 1962	ASCE, Houston Convention
May 1962	ASCE, Omaha Convention
October 15-19, 1962	ASCE, Detroit Convention

Non-ASCE Meetings

February 22-28, 1959	NATIONAL ENGINEERS WEEK
April 13-15, 1959	ASME Hydraulic Conference, University of Michigan
June 15-19, 1959	American Society for Engineering Education, Pittsburgh, Pennsylvania
June 17-20, 1959	National Society of Professional Engineers, New York City
August 1959	International Association for Hydraulic Research, Montreal, Canada

Deadline dates for Newsletter contributions: June issue - April 15.

THE HYDRAULICS DIVISION NEWSLETTER

Your editor sincerely appreciates the cooperation of the various members in furnishing timely and pertinent information for the Newsletter. You are urged to continue use of the Division Newsletter for announcements, inquiries, personnel news, committee reports, surveys and other items of interest to Division members.

GUY L. ARBUTHNOT, JR.
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Vicksburg, Mississippi
Newsletter Editor